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Drainage Correlation Research Project

INTERIM REPORT #5

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ANALYSIS OF THE SOIL CONSERVATION SERVICE
PROJECT FORMULATION PROGRAM - HYDROLOGY

by

Orrin A. Ferris and T. T. Williams

ENGINEERING

Research Laboratories

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MONTANA STATE UNIVERSITY, BOZEMAN



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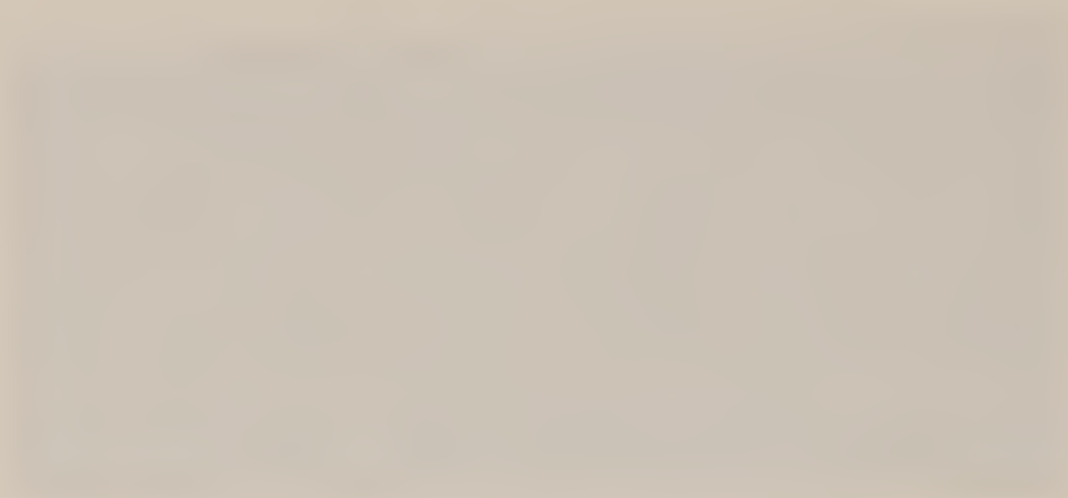
Montana State University, Bozeman

Prepared for

Montana State Highway Department

and

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Chapter I

INTRODUCTION

Inundation of river flood plains by occasional flood discharges has been a threat to the life and property of man since the dawn of civilization. Man has long sought means of predicting floods and averting their danger. Modern man's need for quantitative information concerning flood flows becomes even more pressing as he alters natural waterways with structures, realignments, and diversions.

A major problem continually faced by highway designers, urban planners, and watershed management engineers is that of determining the frequency of peak flood discharges from stream and river basins. For large river basins (in excess of 100 square miles) flood frequency information is generally available by virtue of long term records of precipitation, stream flow, etc., which have been collected by various public agencies (including the U. S. Environmental Science Services Administration, Geological Survey, Bureau of Reclamation, Agriculture Research Service, and others). For watersheds smaller than 100 square miles in area there is generally a shortage of hydrologic data. Only within the past few years has significant research been directed toward a study of flood frequencies on basins of this size.

The "Drainage Correlation Research Project", which was initiated by the Department of Civil Engineering and Engineering Mechanics, Montana State University, Bozeman, Montana, in 1963, addresses itself to the problem of predicting the frequency with which a peak discharge of given magnitude may be expected on small watersheds in Montana. This investigation is sponsored by the Montana State Highway Department and the U. S. Bureau of Public Roads. The work is being done under the direction of

Theodore T. Williams, Associate Professor of Civil Engineering and Engineering Mechanics at Montana State University.

The Drainage Correlation Research Project is a two phase study:

a) to determine flood frequency relationships on watersheds smaller than 100 square miles from long-term climatological data already in existence; and b) to make comprehensive hydrologic studies of four small watersheds in eastern Montana. The watersheds selected for study are Bacon Creek in Wheatland County, Duck Creek in Prairie and McCone Counties, Hump Creek in Sweet Grass County, and Lone Man Coulee in Pondera County. Continuous records of climatic factors and streamflow are being obtained from these four basins. A variety of peak-flow prediction techniques, which utilize data such as that being collected, are being examined.

Among the techniques being considered under the second phase of the Drainage Correlation Research Project is one which was developed by the Soil Conservation Service, U. S. Department of Agriculture (SCS). The SCS method possesses many desirable features, and seems to have considerable potential as a tool for predicting flood frequencies. The study reported in this report analyzes the method developed by the SCS; describes the use of a related computer program; tests the program's ability to reproduce actual hydrographs from a selected watershed; and evaluates the possible utility of this method on Montana watersheds.

The investigation reported herein consisted of the analysis, using the SCS method, of a single rain-caused runoff event which occurred on Duck Creek, the largest of the four watersheds being studied by the

Drainage Correlation Research Project. Duck Creek is an ephemeral stream typical of most eastern Montana streams which drain small watersheds. It is dry most of the time, but occasionally has surface flow after a rain-caused runoff event or during spring snowmelt. In the four years for which data have been obtained at Duck Creek, there have been only a few runoff events of consequence, and only an event which occurred on June 16, 1965 lends itself to an analysis by the SCS method.

A number of events would need to be analyzed before conclusive statements as to the validity of the SCS method could be made. Nevertheless, it is believed that the results of the analysis of this one storm will be a valuable contribution in estimating the applicability of the method to Montana watersheds.



Chapter II

LITERATURE REVIEW

Scientific hydrology is a relatively new area of study. Application of the scientific approach to hydrologic problems has had its greatest growth in this century, and especially in the last thirty years.

Hydrologic measurements were not considered to be of much importance during the early history of the United States. Population centers developed along the rivers and lakes and so no shortage of water was experienced until the westward expansion into the arid and semi-arid regions. The beginning of systematic collection of hydrologic data in this country can probably be taken to be in 1888, when the U. S. Geological Survey under the direction of Frederick H. Newell set up its first river-measurement station on the Rio Grande River at Embudo, New Mexico.

Modern Hydrology

Since about 1930, the increase in the amount of printed information made available in the field of hydrology has increased very rapidly. This indicates something of the increased need there has been in recent years to obtain more data and develop better prediction methods.

The study of hydrology can be subdivided into the two general areas of stochastic and deterministic hydrology. Deterministic hydrology can be further separated into physical or analytical, dynamic and parametric.

Stochastic hydrology attempts to predict future events or reconstruct

past events based on the statistical properties of the known record. White (1967) states that if the value that a variable has implies an element of chance, then it is a stochastic variable. Thus if the record available for some variable (the value of which is random in nature or stochastic) is restricted in time, then the predicted future projection of this record is done by statistical means.

Physical (analytical) hydrology addresses itself to particular specialized problems of the hydrologic process. It does not try to supply any answers to peak discharge, annual yield, or frequency studies. In a sense this branch of hydrology is pure research in that it seeks to establish relationships between variables operating in the hydrologic cycle but does not attempt to solve any related practical problems.

Dynamic hydrology is the term for hydrologic studies involving the dynamic wave theories of fluid flow. These wave theories have been applied to overland flow as well as open channel flow to derive synthetic runoff hydrographs.

Parametric hydrology attempts to discover the relationships among physical parameters that are involved in the particular hydrologic events that are of interest and then to use them to simulate non-recorded events. Amorocho and Hart (1964) list methods of correlation analysis, partial system synthesis with linear or nonlinear analysis, and general system synthesis as methods used in parametric hydrology.

In the prediction of peak runoff from a small watershed, by a method such as that developed by the SCS, both stochastic and deterministic

hydrology come into play. The streamflow itself, being the result of precipitation, is a stochastic process. Watershed characteristics, which modify or affect the streamflow, are, by themselves, physical parameters. Dynamic hydrology must be considered in the flood routing process while parametric hydrology is utilized to find correlations among the various watershed parameters.

A discussion of the technique for predicting flood frequencies which is currently in use by the Soil Conservation Service, is reserved for Chapter III, because it is the basis for this report.

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Chapter III

SOIL CONSERVATION SERVICE RUNOFF PREDICTION METHOD

The Soil Conservation Service method of predicting runoff from ungaged watersheds is characterized by the development of a synthetic unit hydrograph. Under this method the storm characteristics are transformed into a synthesized flood runoff hydrograph by the basin characteristics, the shape of the synthetic unit hydrograph and the baseflow hydrograph. The general procedure which is followed in developing a flood runoff hydrograph is shown by the block diagram in Figure 1.

As shown in Figure 1, the storm runoff volume must first be derived from the storm characteristics and the basin characteristics. This storm runoff volume is then shaped into a storm hydrograph by use of a synthetic unit hydrograph which in turn is derived from the basin and storm characteristics. A baseflow hydrograph then is added to the storm hydrograph as indicated in Figure 1 to produce the flood runoff hydrograph. This sequence of operations involved in producing the flood runoff hydrograph comprises a mathematical watershed model. The general framework of the watershed model has been incorporated into a program for solution by a digital computer. The basin characteristics of a particular watershed must be supplied as input to the computer to transform the general model into a specific model for the watershed in question.

Proposed Use of SCS Method

The proposed way of testing the SCS method* is to use it on an actual

*This method is outlined in the Hydrology section of the SCS National Engineering Handbook (1964).

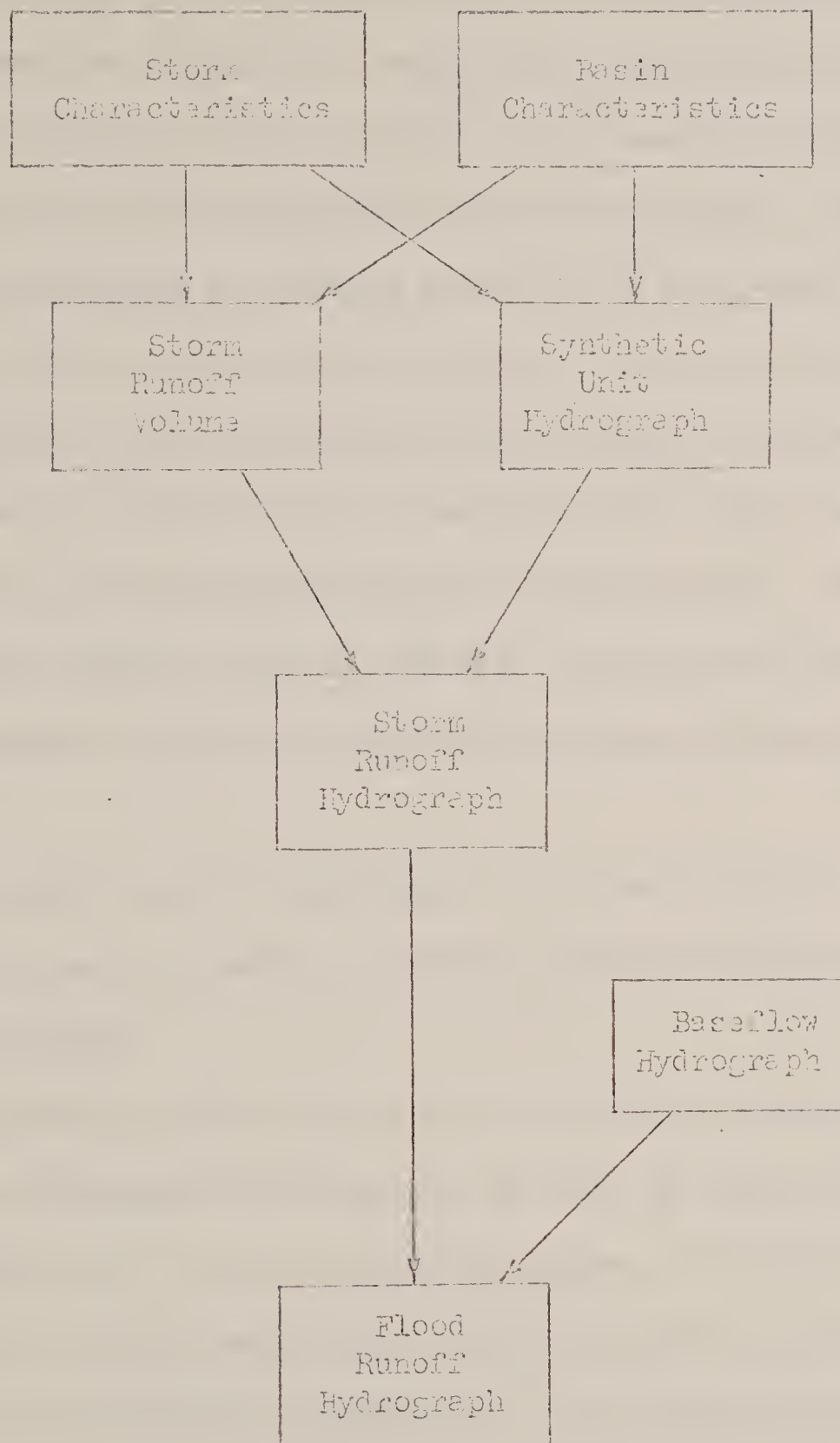


Figure 1: Block Diagram Depicting Soil Conservation Service Runoff Prediction Method



storm where the input (storm) and the output (runoff hydrograph) are known. The procedure should be to supply data characterizing the watershed and storm characteristics as input to the computer program which then uses the watershed model to simulate a runoff hydrograph. The volume of the simulated hydrograph is compared with that of the actual event. If there are major discrepancies in the volumes, the values characterizing the basin parameters may be altered, and the computer program run again. When the volumes are adjusted to be essentially the same, the model is considered to be a valid representation of the watershed. The validity of the SCS method depends upon the ability of the adjusted model to accurately reproduce the actual runoff hydrograph, including the peak discharge.

After the model has been adjusted, it is then possible to route design storms through the model and thereby simulate design hydrographs.

Unit Hydrograph Theory

The unit hydrograph (UH) for a given watershed is defined as the discharge-to-time relationship that yields one inch of runoff from a storm of given duration over the entire watershed area. Unit hydrographs are generally derived from as many actual recorded storms as possible and then generally are assumed to represent the unit runoff functions for all storms of similar durations. Sherman (1932) is given credit for formulation of the unit hydrograph theory while Snyder (1938) first introduced a method for constructing a synthetic UH which may be used for the study of ungaged watersheds. Several other methods for defining synthetic

UH have been developed since Snyder including those by Commons (1942), Mitchell (1948), and Gray (1961).

System Synthesis and Analysis

Where no actual hydrographs are available for a watershed, the SCS method of predicting runoff can be classified as a method of general system synthesis. If an actual unit hydrograph is available, on the other hand, the SCS method would be one of partial system synthesis with linear system analysis.

System synthesis according to Amorochio and Hart (1964) is a method of describing the operation of a physical system with a combination of components that exist in the system and whose functions are known and predictable.

System analysis, on the other hand, is a method by which the relationship between the input and output to the system is established mathematically by measuring only the properties of the input (storm data) and output (runoff) without regarding the nature of the system.

An explanation of the linear and nonlinear properties of runoff prediction methods will follow later in this chapter.

On a gaged watershed since an actual UH is available, it is an analytic (as opposed to synthetic) function since the internal characteristics of the system are not known or synthesized. Even though the actual hydrograph is available, however, synthetic modifications are necessary. Interception of precipitation by vegetation and baseflow characteristics, for instance, must be synthesized. Therefore, even on

a gaged watershed, the SCS method is partially synthetic, and may be classified as one of partial system synthesis with linear analysis.

On an unaged watershed, where it is necessary to use a synthetic UH the entire system is synthetic; thus the classification of general synthesis method applies to the prediction of runoff hydrographs on unaged watersheds.

Functioning of a General Synthesis Method

The system which transforms rainfall to runoff can be thought of as made up of three separate subsystems when the unit hydrograph approach is being considered. 1) The first subsystem, which creates a "rainfall excess" function, modifies the total rainfall input to account for infiltration, interception, and depression storage. The rainfall excess thus computed is the amount of rainfall that is available for runoff. 2) The second subsystem creates a "storm runoff" function by operating on the rainfall excess function. This subsystem makes use of the basin's topographic characteristics. 3) Finally, the third subsystem, which creates the "flood runoff" function uses baseflow information to alter the storm runoff function. The combined effect of these three subsystems is assumed to duplicate the natural processes which occur on the actual watershed being studied. The paragraphs that follow describe in detail these three subsystems as synthesized by the SCS method.

Rainfall Excess: The rainfall excess function (produced by the first subsystem) is given, according to the SCS method, by equation (3).

$$Q = \frac{(P - .2S)^2}{(P + .8S)} \quad (3)$$

where Q is the amount of rainfall excess in inches over the watershed, P is the storm rainfall in inches, and S is defined as the maximum potential difference between P and Q (hence the maximum infiltration capacity) at the time of the storm's beginning.

Equation (3) is based on a hypothesis. If the equation can be shown to be true, then the hypothesis can be assumed to be true. The original hypothesis can be stated as such:

$$\frac{G}{S} \text{ and } \frac{Q}{P} \rightarrow 1 \text{ as } P \rightarrow \infty \quad (4)$$

where G is the actual retention during a storm, S is the potential maximum retention, Q is the direct runoff (or the actual runoff), and P is the total storm rainfall (or the potential maximum runoff). So when flood producing storms are considered, it can be said that:

$$\frac{G}{S} = \frac{Q}{P} \quad (5)$$

Equation (5) cannot be proven mathematically, however, it follows from equation (3) which empirically has been found to be valid.

Since $G = P - Q$, equation (5) can be rewritten:

$$\frac{P - Q}{S} = \frac{Q}{P} \quad (6)$$

The SCS Hydrology Handbook begins the development of equation (3) with

the statement of equation (6).

As P goes to infinity, the ratio $(P - Q)/S \rightarrow 1$ and $Q/P \rightarrow 0$. Keeping in mind that P , Q , and S are total volumes for a storm, $(P - Q)$ finally fills the entire soil profile to its maximum value S as determined for the condition of the watershed at the beginning of a storm. Now, solving directly for runoff volume from equation (6):

$$Q = \frac{P^2}{P + S} \quad (7)$$

A further adjustment of P can be made for the initial abstraction I_a which reduces P by the amount of interception, depression storage, and infiltration at the storm's beginning before runoff occurs. Several relationships can now be revised to allow for the initial abstraction (in effect, this amounts to redefining terms which now have slightly different meanings than the corresponding terms in equations (4) through (7)).

$$P = Q + G + I_a \quad (8)$$

$$S \approx I_a + G = P - Q \quad (9)$$

$$G = (P - I_a) - Q \quad (10)$$

Equation (7) can also be restated:

$$Q = \frac{(P - I_a)^2}{P - I_a + S} \quad (11)$$

Existing data from several watersheds were studied and it was found that

I_a can be taken to be equal to .2S. By substituting this value in equation (11), equation (3) is finally derived.

The SCS has prepared a graphical solution (runoff as a function of precipitation) of equation (3), which is shown in Appendix A. As is evident in Appendix A, precipitation is related to runoff through a family of curves, each curve being drawn for a particular curve number (CN). The curve numbers are a function of S and are given by the relationship:

$$CN = \frac{1000}{S+10} \quad (12)$$

The parameter CN is used instead of S as integer values of S do not produce curves that lend themselves to easy interpolation. The curve numbers, however, make the P vs. Q plot easier to use.

The runoff equation, equation (3), used by the SCS is a relationship between P and Q involving only one parameter, the parameter being S. S can now be empirically related to as many characteristics as is thought to be necessary. The developers of this method found that soil and cover conditions had the most effect on the value of S (and therefore on the CN). The curve number CN is therefore termed the soil cover complex number.

For various combinations of soil type and vegetal cover, soil cover complex numbers have been developed empirically. To determine such a number, small watersheds were found which had only one type of soil and only one type of cover condition. A number of points were plotted, one

point per storm, on a graph of total precipitation in inches versus total runoff in inches. The CN curve that best fits this plot is then taken to be the representative curve number for that soil cover complex. The results of all such plots made by the SCS are summarized in Appendix C.

In using the SCS method, curve numbers (CN) are determined by referring to a table such as that shown in Appendix C, and selecting the CN that corresponds to the appropriate soil type and vegetal cover condition. The soil classification used divides all soils into four groups (A, B, C, and D) according to their permeabilities. The permeability of group A soils is highest and of the D soils, the lowest. The descriptions of the various soil types are given in Appendix B. Musgrave and Holtan, as reported by Chow (1964), have quantified infiltration rates for the four soil groups. Minimum infiltration rates for groups D, C, B, and A are given by the ranges 0 to 0.05, 0.05 to 0.15, 0.15 to 0.30, and 0.30 to 0.45 inches per hour, respectively.

A vegetal cover condition is described as some natural or cultivated condition. For instance the cover may be described variously as fallow, contoured small grain in poor condition, pasture in fair condition, or woods in good condition.

The effect of the antecedent moisture condition (AMC) is accounted for by adjusting the soil cover complex number previously calculated. The curve number was originally developed with average AMC being assumed. However, for conditions of very dry and very wet antecedent soil moisture conditions, type I and type III, respectively, are used. The type II

condition is considered to be the average condition. Whether the AMC calls for type I or type III treatment is determined by the guide lines in Table 1. The limits on the dormant season apply to unfrozen ground and no snow cover.

After the correct type of AMC is determined, there are standard curves showing the adjustments to be made on the soil cover complex numbers as shown in Figure 2. Values for the adjusted CN can be taken at points between lines shown for types I, II, and III if it is felt that the AMC are sufficiently well known.

Storm Runoff: The second subsystem (that which produces the storm runoff function) is assumed to be a linear convolution by unit hydrograph theory. The purpose of the convolution process is essentially to convert a rainfall excess function of a certain volume into a runoff function having the same volume. The convolution integral describing this runoff function $q(t)$ is given by:

$$q(t) = \int_0^{t'} u(t - \tau) i(\tau) d\tau \quad (13)$$

The kernel function $u(t - \tau)$ is the instantaneous unit hydrograph (the unit hydrograph which theoretically results from a rainfall excess occurring instantaneously on all parts of a watershed), $i(\tau)$ is the rainfall excess input function, and t' (the limiting time for τ) is a function of t_0 , t_0 being the duration time of the input function (storm). For $t < t_0$, $t' = t$ and when $t \geq t_0$, $t' = t_0$.

The basic convolution process is shown graphically in Figure 3.

AMC	Total 5-day Antecedent Rainfall	
	Dormant Season (inches)	Growing Season (inches)
I	0.5	1.4
II	0.5 - 1.1	1.4 - 2.1
III	1.1	2.1

Table 1: Types of Antecedent Moisture Conditions

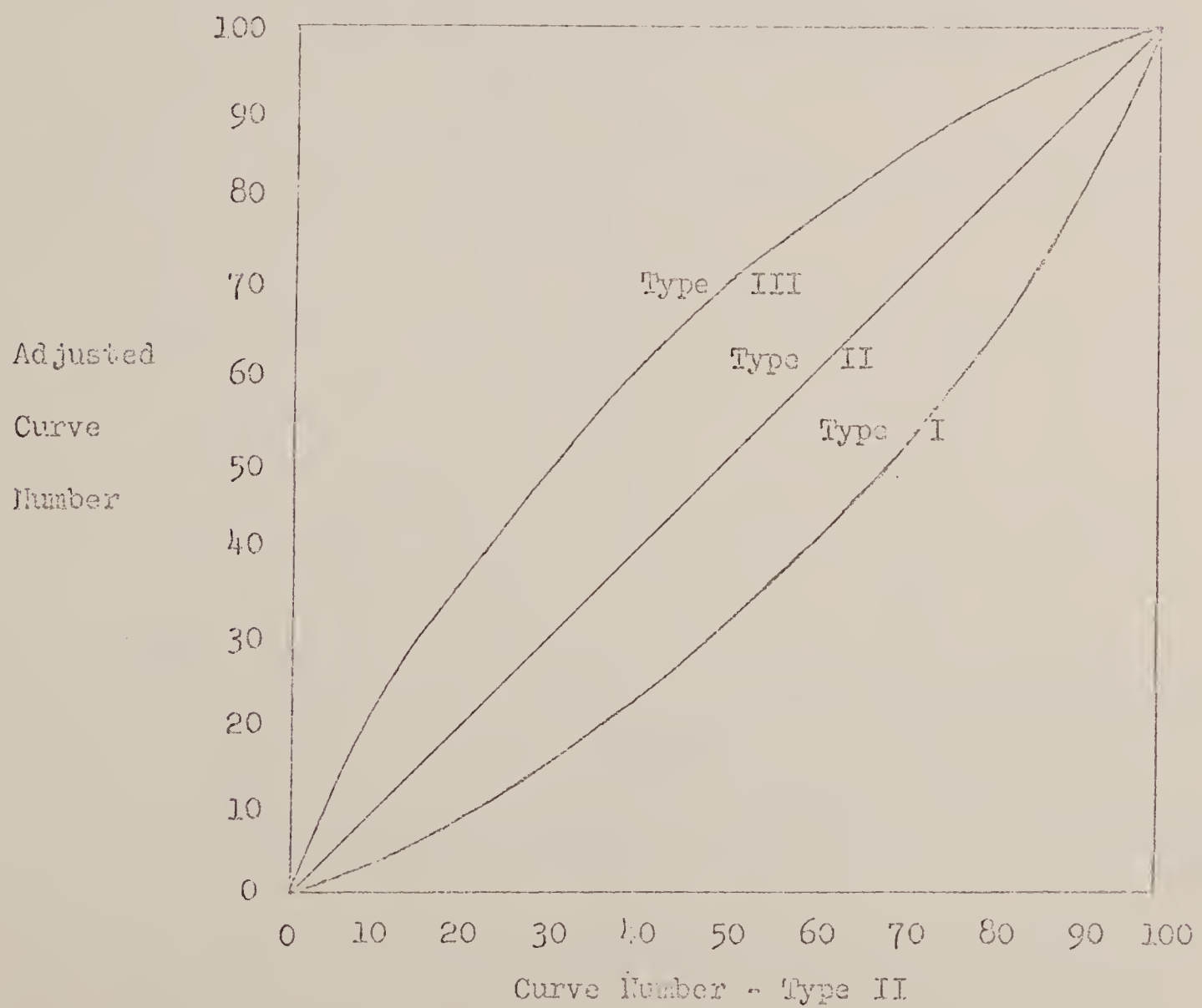


Figure 2: Adjustment of Curve Number Value for Types of AMC



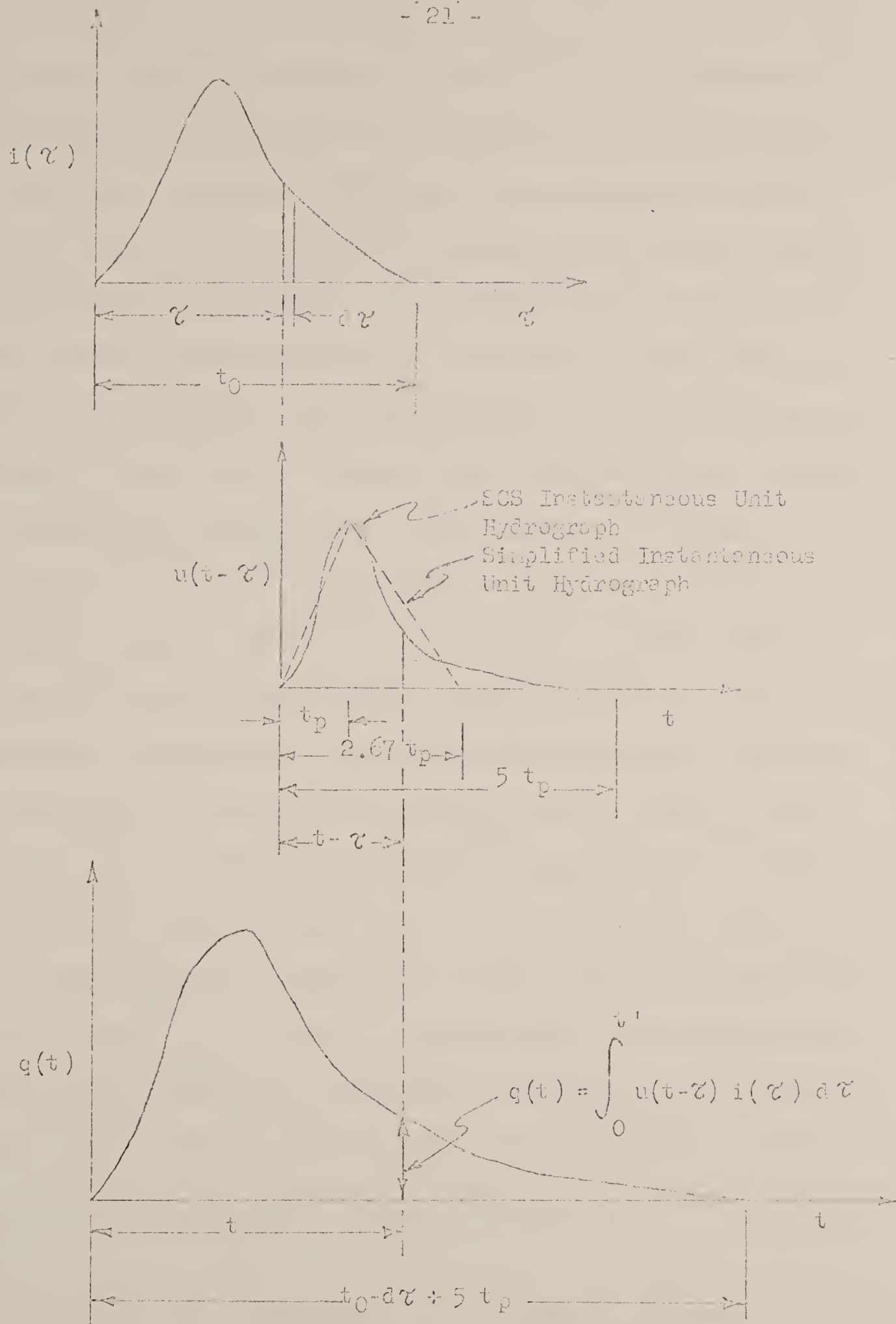


Figure 3: Convolution of Rainfall Excess $i(\tau)$ and Instantaneous Unit Hydrograph $u(t-\tau)$

Starting from the excess function $i(\tau)$ versus τ , each incremental volume of rainfall is multiplied by the instantaneous UH (IUH) and is summed in the right time sequence to yield the storm runoff function $q(t)$ versus t . The instantaneous unit hydrograph shown is that used by the SCS when developing a synthetic unit hydrograph for an ungaged watershed. This synthetic UH was developed for the SCS by Victor Mockus and reported in the SCS National Engineering Handbook (1964). The triangular IUH shown with a dotted line is normally used, because of its simplicity for runoff calculations made by hand. However, when the computer is utilized, the more exact curvilinear IUH is equally easy to apply.

The rainfall excess function $i(\tau)$ can be used directly with the IUH to produce the runoff function $q(t)$. However, the way in which values of $i(\tau)$ should be calculated is not obvious from the earlier development of equation (3). The volume of rainfall excess i (which is equivalent to the volume of runoff q) for the time increment $d\tau$ is equal to the value Q given by equation (3) at time $\tau + d\tau$ minus Q by equation (3) at time τ . The volume of precipitation P is the total precipitation of the storm up to time $\tau + d\tau$ and τ , respectively. The increment of Q , (dQ), caused by the storm of $d\tau$ length must be given as a difference of Q 's at times $\tau + d\tau$ and τ . This is true since the runoff equation is based on the entire storm (the initial abstraction must be accounted for only once).

The base time for the curvilinear UH in Figure 3 is taken to be 5.00 times the time to peak. This value of base time is arbitrary but does

not incorporate appreciable error in the results as the corresponding values of discharge are very small. The peak flow rate for the curvilinear UH is the same as that for the triangular version whose base time (determined mathematically) is found to be 2.67 times the time to peak (t_p). The triangular hydrograph base time of $2.67 t_p$ is used as it gives the same value of runoff volume as the curvilinear version, the volume being represented by the area under the curve. The peak flow rate q_p is derived using the triangular shaped UH and is given by the general equation

$$q_p = \frac{KAQ}{t_p} \quad (14)$$

with
$$t_p = D/2 + L = D/2 + 0.6 t_c \quad (15)$$

and
$$t_b = 2.67 t_p \text{ for triangular unit hydrographs} \quad (16)$$

and
$$t_b = 5.0 t_p \text{ for curvilinear unit hydrographs} \quad (17)$$

where K is a constant whose value depends on the units used, A is the area, Q is total volume of runoff, t_p is time to peak, t_b is the base time, D is storm duration time, t_c is the time of concentration, and L is the basin lag time. K is equal to 484 when Q is in inches, A in square miles and t_p , L and D are in hours. Peak flow is then in cubic feet per second.

For the example shown in Figure 3, where the convolution integral generally cannot be solved exactly, actual practice dictates that the

storm input be broken up into small increments ΔZ . The storm duration D of equation (15) is then equal to ΔZ . Equation (14) together with either (16) or (17) is sufficient to describe the runoff hydrograph from a subwatershed due to an incremental storm. The hydrograph can be given the triangular or curvilinear shape as desired.

Flood Runoff: The third subsystem (that which produces the flood runoff function) is the simple procedure of adding the baseflow hydrograph to obtain the resulting flood hydrograph. The baseflow hydrograph may be obtained by any of the standard textbook methods as long as the same procedure is used consistently.

Channel Flood Routing

When a watershed is divided into subwatersheds for the purpose of predicting runoff more accurately, it is necessary to route hydrographs through the natural channels. The routed hydrograph (outflow hydrograph from a channel) is then in a proper form to be added to another routed hydrograph or to another subwatershed runoff hydrograph, after which it may be routed through a second reach of channel.

The method chosen by the SCS for channel flood routing is termed the "convex method" after the name applied to the mathematical principle upon which it is based.

The convex method is based on the principle that when a flood passes through a reach of natural channel there is a time interval such that

$$\text{if } I_1 \geq O_1, \text{ then } I_1 \geq O_2 \geq O_1 \quad (18a)$$

$$\text{if } I_1 \leq O_1, \text{ then } I_1 \leq O_2 \leq O_1 \quad (18b)$$

where I_1 and O_1 are the rates of inflow and outflow respectively at time $t = t_1$, and O_2 is the rate of outflow at time $t = t_1 + \Delta t = t_2$. Note that I_2 (inflow rate at t_2) does not enter into this method. If Δt is chosen as specified then I_1 , O_1 , and O_2 form a "convex set". From convex set theory we can then write the working equation

$$O_2 = (1 - C) O_1 + CI_1 \quad (19)$$

where C is a parameter with the range $0 \leq C \leq 1$.

From the definition of a convex set, it can be said that whenever any two points x and y are in a convex set L , then every point on the line connecting them is also in L . A formal definition from set theory given by Charnes and Cooper (1961) states that: a set of points L is convex if, whenever $x, y \in L$, then $ux + (1 - u)y \in L$ for all $0 \leq u \leq 1$.

The limits on the value of C insure that equation (19) meets the conditions of the equations (18a) and (18b). It follows that:

$$C = \frac{O_2 - O_1}{I_1 - O_1} \quad (20)$$

It can also be shown from Figure 4 that:

$$\frac{O_2 - O_1}{\Delta t} = \frac{I_1 - O_1}{K} \quad (21)$$

where K is a parameter with time units not yet having any physical mean-

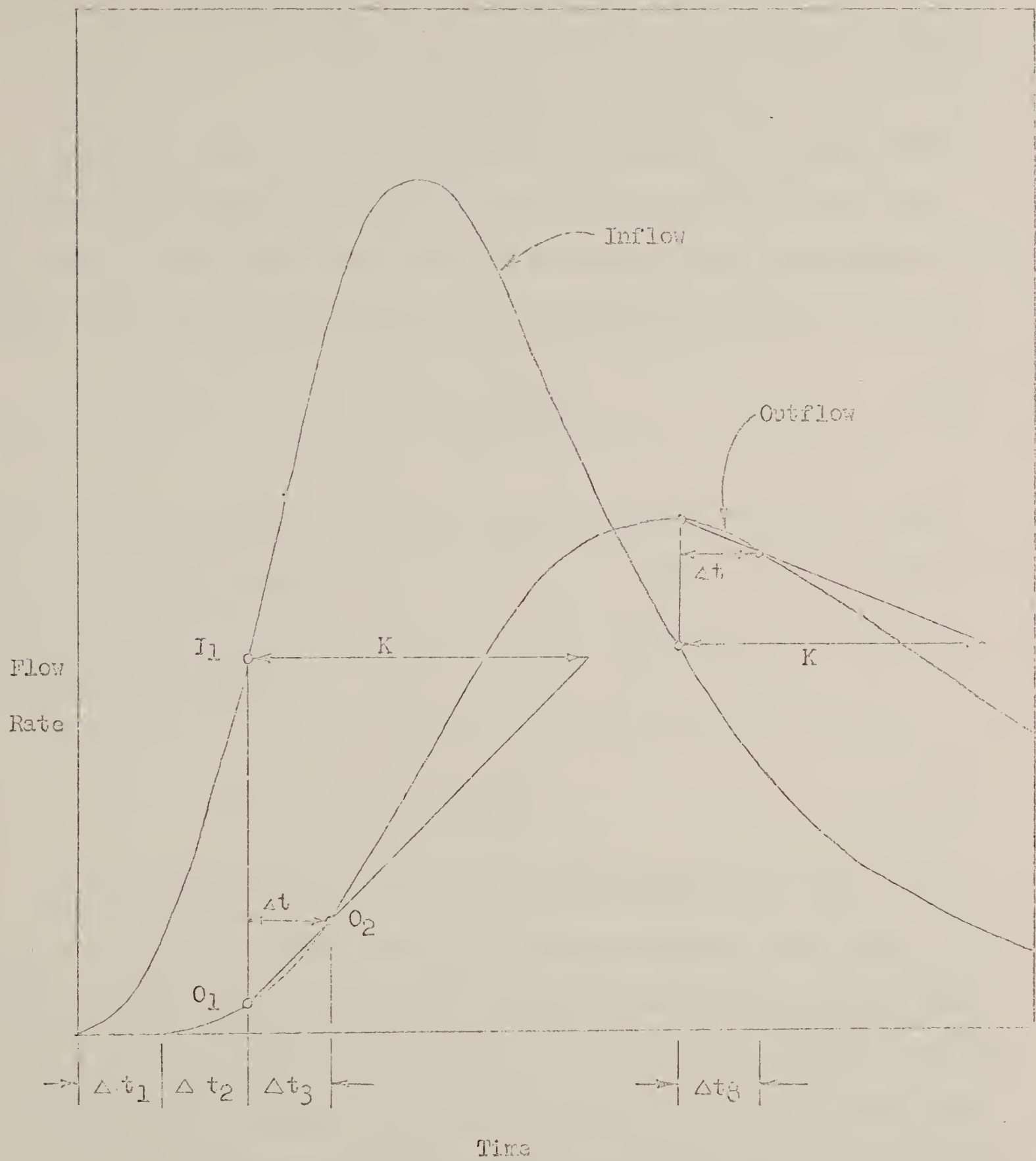


Figure 4: Relationship between Δt and K

ing. From equations (20) and (21) it can be seen that:

$$\Delta t = CK \quad (22)$$

If use of this method is to be made on ungaged watersheds, some empirical relationships need to be developed for evaluation of C and K. This has been done by the SCS from knowledge gained from study of gaged watersheds. It was found that equation (23) expresses the value

$$C = \frac{\bar{V}}{\bar{V} + 1.7} \quad (23)$$

for C very well except at extreme values of \bar{V} (<0.5 and >10.0), where \bar{V} is the average stream velocity in feet per second. K was also found to be the travel time for the reach. If K is in hours and L is the reach length in feet, then:

$$K = \frac{L}{3600 \bar{V}} \quad (24)$$

It is obvious from the empirical relationships for C and K that as the water surface elevation changes, varying velocities will cause C and K to vary with time. A procedure for determining \bar{V} was sought so that C and K could be assumed constant for any given routing problem. It was found that if the average velocity \bar{V} was based on the stream flow velocities for all discharges of the inflow hydrograph greater than one-half of the peak discharge, the resulting outflow hydrograph was nearly the same as if it were calculated using varying C and K.

The value of Δt to be used is then a product of C and K and remains constant for the entire routing. The value of Δt may turn out to be inconvenient fractions of an hour and therefore difficult to use. SCS practice is to choose a convenient $\Delta t'$ and calculate a new C , called C' , such that routing by equation (19) gives identical results. The adjustment is

$$C' = 1 - (1 - C) \Delta t' / \Delta t \quad (25)$$

Routing is now accomplished by rewriting equation (19) to give the result:

$$O_2 = (1 - C') O_1 + C' I_1 \quad (26)$$

Reservoir Flood Routing

Runoff hydrographs are routed through reservoirs by the storage indication method. This method is based on the continuity equation shown here in its differential form,

$$i(t) = q(t) + \frac{dS}{dt} \quad (27)$$

where $i(t)$ is the rate of the inflow at time t , $q(t)$ is the rate of outflow at time t , and dS/dt is the rate of change in storage. The basic working equation is written:

$$(I_1 + I_2) \Delta t / 2 - (O_1 + O_2) \Delta t / 2 = S_2 - S_1 \quad (28)$$

where I is inflow, O is outflow, S is storage, t is a time increment, subscripts 1 and 2 are for $t = t_1$ and $t = t_1 + \Delta t = t_2$ respectively.

Reduction of this gives:

$$(I_1 + I_2 - O_1) \Delta t + 2S_1 = O_2 \Delta t + 2S_2 \quad (29)$$

Defining constants C_4 and C_5 as:

$$C_4 = (I_1 + I_2 - O_1) \Delta t + 2S_1 \quad (30)$$

$$C_5 = O_2 \Delta t + 2S_2 \quad (31)$$

It then follows that from equation (29):

$$C_4 = C_5 \quad (32)$$

C_4 is first evaluated from known values, then set equal to C_5 . This value of C_5 is then substituted into equation (31). There is one set of values (O_2 and S_2) on the storage-discharge curve for the reservoir that satisfies this linear equation. This procedure constitutes one step in solving the continuity equation by numerical approximation. To route a hydrograph through a reservoir this process must be repeated until flood flows have nearly diminished to the original backflow.

Systems Classification of SCS Method

The runoff dynamics of an actual watershed might be described as a distributed, non-linear, time variant system. Many of the models constructed to describe this system, however, are classified as lumped, linear, and time invariant. Early models with these assumptions were those developed by Sherman (1932) and Bernard (1935). Sherman developed the unit-graph concepts often called unit hydrograph theory. Bernard is

given the credit for implementing use of the distribution graph (a bar graph comparable to a unit hydrograph except that runoff volumes are expressed as percent of total runoff versus time).

A distributed system, in the field of hydrology, is defined as one with time and space variation in rainfall intensities. A lumped system or model is one which allows only time variation but no space variation in rainfall intensity. This is to say that rainfall intensity is uniform over the watershed area under consideration at any given time.

The SCS method in its basic form (as currently explained) would be classified as a lumped system. By using the SCS runoff prediction method in conjunction with reservoir and channel flood routing techniques, the resulting system will in part be distributed. Chow (1967) describes this type of system as a "distributed system of lumped-system models". On each of several subwatersheds the rainfall intensity is uniform, or each of the subsystems is lumped. However, at any given time, the intensity can vary from subwatershed to subwatershed and is in this sense distributed.

A system is linear if the unit responses of various input magnitudes are identical. The watershed runoff function is linear if one unit hydrograph is sufficient to accurately describe all storms of a given duration occurring on a given watershed. This is never the case, however, as Childs (1958) and others have shown. Childs states that the larger the flood, the higher the peak and the shorter the time to peak on the unit hydrograph. He demonstrates this fact by showing four unit hydrographs

of three-hour storms on Naugatuck River at Thomaston, Connecticut. The peak of a major flood has more than double the peak derived for a minor flood and about two-thirds of the time to peak. Obviously, the variance of the storm pattern or rainfall excess function affects the unit hydrograph shape. Non-linearity is then the case when the derived unit hydrograph varies with storm pattern and total rainfall excess amount.

A system that gives the same response for a given input regardless of time is a time invariant system. The subwatershed portion of the SCS model is based on unit hydrograph theory and is therefore time invariant. This is not too restrictive an assumption as the rainfall input to the subwatershed runoff model is preadjusted for soil and cover conditions, antecedent moisture, and time of year. These are the main items giving the runoff function (system response) a time variant nature.

Spatial and temporal variations in rainfall distribution can be accounted for with the SCS approach. These variations are simulated by dividing a watershed into several subwatersheds and by making the time increment on the precipitation versus time input graph small enough so that the graph nearly matches the actual curved relationship. The total amount of rainfall can be different for each subwatershed. The storm pattern can have an infinite number of shape variations. Spatial variation is not totally accounted for since there is still no spatial variation allowed within the subwatershed; thus spatial variation is only partially simulated. The subwatersheds can, however, be made as small as is practical for the time and expense that can be spent in the analysis.

The time of storm beginning and duration can also be designated as necessary to conform to an actual storm. Therefore, temporal variation is also partially simulated.

Chapter IV

SOIL CONSERVATION SERVICE PROJECT FORMULATION PROGRAM - HYDROLOGY

The Soil Conservation Service, in 1963, obtained (from a private consulting firm known as C. E. I. R., Inc.) a digital computer program designed to solve the "runoff hydrograph prediction" relationships described in Chapter III. The computer program, which was written in the Fortran language for the IBM 7090 computer system, is entitled "Project Formulation Program - Hydrology" and is a mathematical model of a watershed experiencing precipitation, surface runoff, and channel flow. The program can route an unlimited number of hypothetical or actual storms through a watershed having as many as 60 structures and 120 cross-sections described.

The program has gone through several revisions, most recently in October, 1967. It is now written in Fortran IV-G and utilizes the IBM 360/40 computer system. The most recent instructions for using the Project Formulation Program (PFP) were published by C. E. I. R., Inc. (1964) and the SCS (1965).

The PFP consists essentially of a series of subroutines which may be solved in a variety of combinations and sequences; a schematic overview of the PFP is shown in Figure 5. The particular sequence to be followed in a given problem is determined by instructions to a "Standard Control" routine, which calls the subroutines in the proper order, and provides the additional information necessary to perform the work of the subroutine. The entire program is monitored by an "Executive Control", which describes the particular storm under consideration.

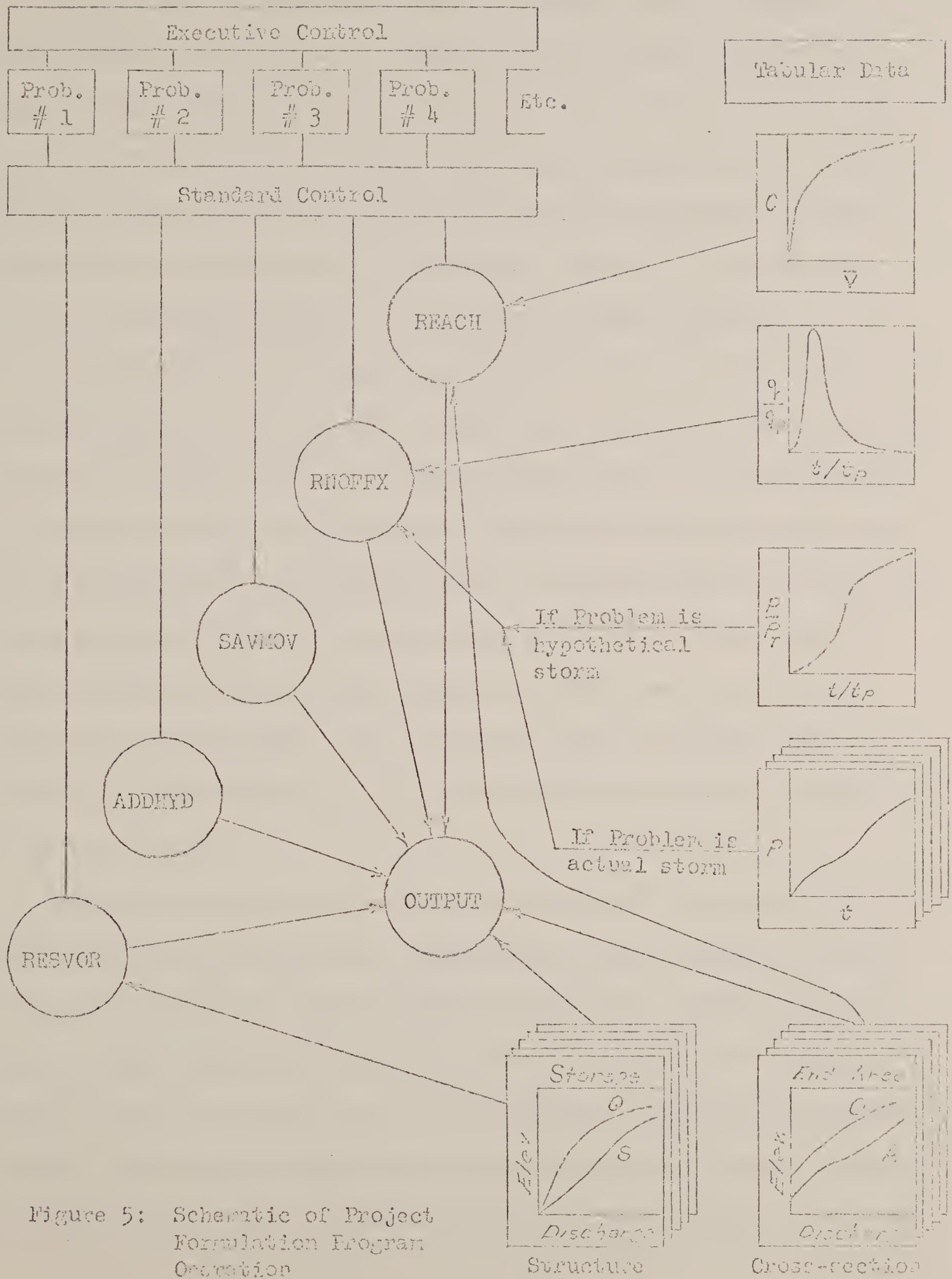


Figure 5: Schematic of Project Formulation Program Operation

To formulate a PFP problem, it is first necessary to read in several tables, as indicated on Figure 5, some general and some specific for the particular watershed. After this is done, Standard Control cards are read in to describe the logical sequence in which flood routings through the stream reaches and structures should be performed. This sequence of operations is, of course, identical to the sequence that the hydrologist follows in solving the problem manually.

To reflect a future change in a watershed, any portion of the Standard Control can be modified at any time by addition, alteration, or deletion of watershed data. Executive Control cards are then read into the computer to describe each of the alternative storm situations that are to be analyzed. An infinite number of problems may be set up and run by the Executive Control for a given set of watershed characteristics. Each directive of the Executive Control specifies the storm precipitation, its starting time, the antecedent moisture condition under which it is to be analyzed, and the portion of the watershed through which it is to be routed.

Each subwatershed described may have different total rainfall amounts, different storm starting times, different storm time durations and patterns, and varying antecedent moisture conditions. A given storm situation is then applied to the Standard Control to effect a solution. Additional storms may then be set up and solved. As has been seen, the Standard Control can at any time be changed to reflect proposed changes on the watershed. After a change, any hypothetical or actual storm can

again be applied to the modified watershed to evaluate the changes made in the outflow hydrograph.

PEP Subroutine Arrangement

The SCS program has a total of twelve subroutines (seven of which are computational). Their relationships to the main program and to other subroutines are shown in Figure 6.

The seven computational subroutines are called RECFPX, RESVOR, REACH, ADDHYD, OUTPUT, INTERP, and PEAK. They, respectively, calculate a runoff hydrograph for a subwatershed; route an input hydrograph through a reservoir by the storage-indication method; route an input hydrograph through a channel reach by the convex method; add two hydrographs; print and/or punch the output options specified; adjust the time increment for RESVOR, REACH, and ADDHYD; and calculate coordinates of the highest peaks (to a maximum of ten) of a hydrograph.

The other five subroutines are called SAVMOV, UPDATE, SETUP, READ 5, and READIN. SAVMOV gives the writer of Standard Control the ability to move an entire hydrograph from one storage location to another and to save it there until needed later. UPDATE causes the Standard Control listing currently in force to be printed out. This would be used after some alteration of Standard Control had been made. SETUP has the overall control of the computational subroutines. READ 5 and READIN cause the program to read in the data which describe the physical features of the watershed and the properties of the storm being simulated.

Tabular Data

Several types of data that are necessary for the operation of the

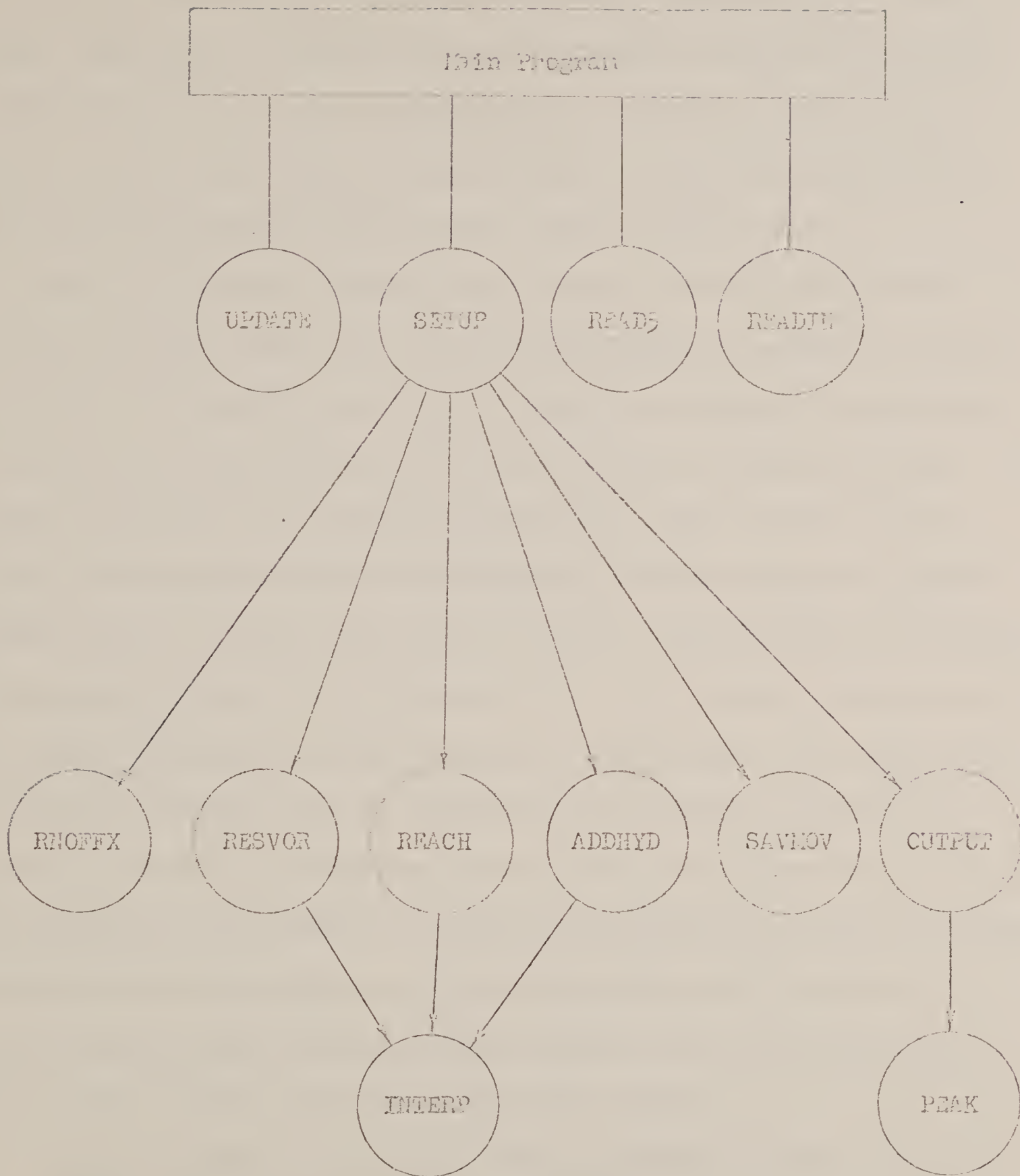


Figure 6: Subroutine Scheme for Soil Conservation Service
Project Formulation Program - Hydrology

Project Formulation Program can be supplied to the computer in tabular form. These data are values for the relationship between the routing coefficient C and average velocity \bar{V} , the curvilinear unit hydrograph, storm patterns, the stage-storage-discharge relation for reservoirs and the stage-area-discharge relation for channel cross-sections.

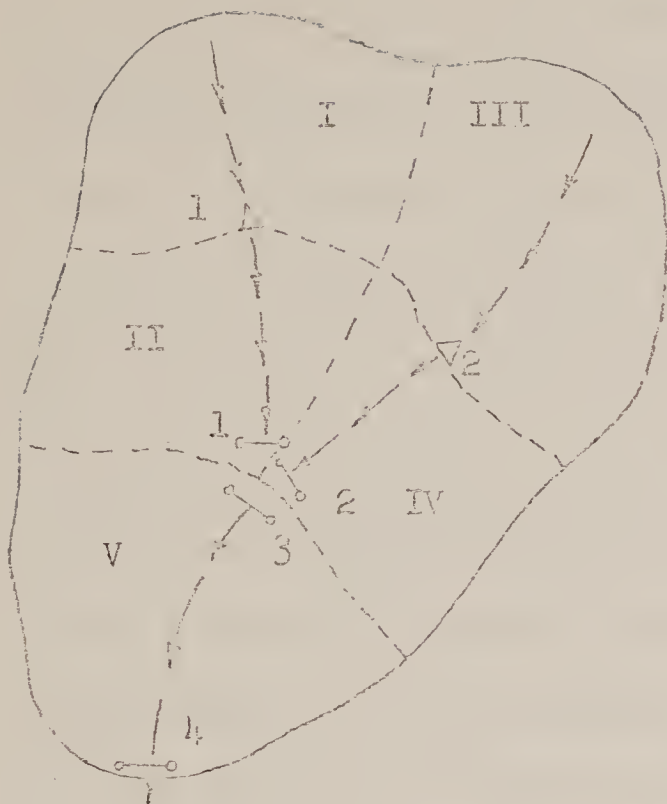
The $C - \bar{V}$ graph is tabulated for computer use in velocity increments of 0.2 feet per second. The unit hydrograph is supplied in dimensionless form as q/q_p versus t/t_p . Design storm patterns are described in cumulative, dimensionless form with coordinates of P/P_T and t/t_p where P_T is the storm's total precipitation. Actual storms can be read in either dimensioned or dimensionless form but always as a cumulative graph. If actual dimensions are used, rainfall depth and duration in Executive Control are to be given as 1.0. If, however, dimensionless coordinates are used for actual storms in tabular data, the actual values of depth and duration must be supplied in the Executive Control. The structure and cross-section data can be given in any desirable increment of elevation. This should be a small enough increment so that the assumed straight line between points is nearly coincident with the curved relationship from which the points were selected. The elevation increment can be varied at will throughout these tables.

The way in which these tabular data are utilized by the Executive and Standard Control is outlined by the flow chart in Figure 5. The problems posed by the Executive Control are individual, distinct storms, an infinite number of which can be processed by the PFP. After problem

No. 1 has been worked by the Standard Control through its various sub-routines, problem No. 2 is then described by the Executive Control and solved. This continues until all stated problems have been solved. These subroutines in turn use the tabular data (shown in squares on Figure 5) to complete their function. REACH needs the C - V and the cross-section graphs to complete its task. The cross-section information is needed whether OUTPUT is called or not. RHOEFFX uses the UI and either a design or actual storm P - t graph. SAVMOV and ADDHYD do not make use of the tabular data. RESVOR must call the structure data to perform its routing problem. All five of the working subroutines may have to call OUTPUT (if specified by standard control) before returning to Standard Control.

Standard Control

Standard Control is an ordered set of instructions to the computer that outline the sequence of operations that simulate runoff from a watershed area. These operations, which have already been outlined, are done by the subroutines RHOEFFX, RESVOR, REACH, ADDHYD, and SAVMOV. Figure 7 shows a sample watershed, with two reservoirs and two main stream branches, which is subdivided into five subwatersheds. A table of the corresponding Standard Control listing for this sample watershed is also given. The structure and cross-section locations are numbered so that the reader can follow through the Standard Control to see that it adequately describes the way in which rainfall excess is converted into runoff. There are, however, some assumptions in this connection which make the simulation



- II Subwatershed area
- △ Structure location
- Cross-section location
- Stream course
- Subwatershed boundary

Area Subroutine	Cross- Section	Structure	Hydrograph Storage Locations			Output Options			
			Input #1	Input #2	Output	Hyd	Elev	Peak	Vol
I	RNOFFX	01			6	1		1	1
	RESVOR	01	6		7	1	1	1	1
	REACH	001	7		5	1	1	1	1
II	RNOFFX	001				1		1	1
	ADDHYD	001	5	6	7	1	1	1	1
	SAVMOV	001	7		1				
III	RNOFFX	02			6	1		1	1
	RESVOR	02	6		7	1	1	1	1
	REACH	002	7		5	1	1	1	1
IV	RNOFFX	002			6	1		1	1
	ADDHYD	002	5	6	7	1	1	1	1
	SAVMOV	002	7		5				
	SAVMOV	002	1		6				
	ADDHYD	003	5	6	7	1	1	1	1
V	REACH	004	7		5	1	1	1	1
	RNOFFX	004			6	1		1	1
	ADDHYD	004	5	6	7	1	1	1	1

Figure 7: Sample Watershed with Corresponding Standard Control for the Soil Conservation Service Project Formulation Program.

much easier without adding substantial error to the operation. For example, the outflow hydrograph from structure 1 is routed through area II as if no additional runoff were being supplied from area II. Now, there exists an outflow hydrograph from the channel reach in area II at section 1. To this hydrograph is then added the runoff hydrograph from area II as if it had not arrived at section 1 by way of the channel system. This assumption is, of course, not physically the case but soon after leaving the upstream subwatersheds the channel hydrograph is very large compared to the local area runoff so that the error resulting from this assumption is considered small.

Computer locations for hydrograph storage are shown on the table included in Figure 7. There are seven different locations supplied where all the coordinates of points from a hydrograph can be stored (up to 300 points). These points are all described by a time and discharge value. All subroutines have an input and an output hydrograph except ADDHYD which has two input and one output hydrographs. The numbering of storage locations as shown is set and consistently used except for the SAVMOV subroutine which can then be used to place a hydrograph in the correct location for the subsequent operation. This rigid numbering system is used so that fewer errors will be made by a technician preparing the Standard Control.

All the possible output options are also shown in Figure 7. These include printing out all the values of the hydrograph, the corresponding water surface elevations, up to the ten highest peak discharge values and

their times of occurrence, and the total volume under the hydrograph given in three sets of units, cfs-hrs, acre-feet, and inches of runoff over the watershed.

Other information than that shown must also be supplied. Area in square miles, runoff curve number CN, and time of concentration in hours must also be given with the RNOFFX command. RESVOR must also know the surface elevation of the reservoir at time $t = 0$. Length of a reach in feet and a routing coefficient (if one is to be supplied instead of being calculated by the program) are needed to complete the REACH subroutine. ADDHYD and SAVHYD are complete as shown in Figure 7.

With regard to the numbering of stream channel cross-sections and control structures, numerical values may be given in any order, although a logical sequential order is easier for others to follow. However, the way in which these numbers are used in Standard Control is very important as will be seen when the use of the Executive Control is explained. The important thing to remember is that any number used more than once must appear consecutively in the Standard Control listing. That is, if cross-section 001 is used as it is in Figure 7, the four times that it appears must follow one another without interruption by other numbers.

Executive Control

Executive Control is a set of commands that have the over-all control of the computer solution to stated hydrologic problems. As indicated earlier, each Executive Control directive specifies the properties of the hypothetical or actual storm that is to be routed. Storm

properties accounted for in this model include storm pattern, total rainfall amount, storm length, and starting time. The properties may vary from subwatershed to subwatershed but are assumed uniform over each individual subwatershed.

The storm pattern is generally read into the computer with a dimensionless cumulative rainfall amount ordinate and a dimensioned time coordinate. However, it is possible to read in both coordinates with either dimensioned or dimensionless scales. The dimensionless values are then transformed to dimensioned values by specifying the total rainfall amount, the total storm length that applies to the particular case, or both. This flexible arrangement is useful for applying hypothetical storms to the system. In the case of actual storms, cumulative graphs with the correct time and precipitation dimensions can be read in directly.

INCREM, COMPUT, ENDCMP are additional statements used in Executive Control which simply are shorthand for increment, compute, and end of computations, respectively. They are not subroutines but simply commands that are used by the main program to carry out the desired control sequences.

Examples of Executive Control: Several example problems are illustrated in Table 2 to demonstrate the flexibility of the Executive Control. Problem No. 1 breaks the watershed into three subwatersheds each receiving different amounts of rainfall. Rainfall Table No. 1 would be given in dimensionless depth units and time in hours since the amount of rainfall is specified and 1.00 is given under rainfall duration. The

Prob	Command	Hyd t hrs.	From		Through		Starting Time hrs.	Rainfall			AMC
			Sec.	Str.	Sec.	Str.		Depth (in.)	Duration (hours)	Table (no.)	
1	INCREM	1.0		01	001		0.0	4.15	1.00	1	2
	COMPUT			02		02	0.0	3.50	1.00	1	2
	COMPUT		002		004		0.0	4.29	1.00	1	2
	ENDCMP										
2	COMPUT			01	001		0.9	4.15	1.00	3	1
	COMPUT			02		02	0.0	3.50	1.00	3	1
	COMPUT		002		004		1.7	4.29	1.00	3	1
	ENDCMP										
3	INCREM	0.5		01	001		0.0	1.00	1.00	2	3
	COMPUT			02	004		0.5	1.00	1.00	2	3
	ENDCMP										
4	INCREM	1.0		01	004		0.0	4.50	1.00	4	2
	COMPUT										
	ENDCMP										
5	COMPUT			01	004		0.0	3.00	1.00	5	2
	ENDCMP										

Table 2: Example of Executive Control on Sample Watershed.

antecedent moisture condition (AMC) is type II or normal. Starting time is given as zero on all subwatersheds.

Problem No. 2 is similar to No. 1 except that storm movement is accounted for by varying the starting time on the various subwatersheds. Also the storm pattern is considered to be different in shape or length as rainfall Table No. 3 is used. The AMC is type I or very dry.

Problem No. 3 is an actual storm that is taken as uniform in amount over the entire watershed. Its starting time varies for two subareas and a new Table (No. 2) describes the storm patterns which is given in true units of inches and hours since the factors used for rainfall depth and duration are both given as unity. Problems 4 and 5 are design storms and are uniform over the entire area in rainfall amount and starting time. The rainfall depths given would correspond to some calculated design storm amount for a given frequency storm for the area in question. Rainfall Tables 4 and 5 might be storm patterns for 12 hours and 24 hour storms. A normal AMC would always be assumed for design storms.

INCREM states the time increment in hours that the computer should use in developing the hydrographs. COMPUT instructs the computer to compute what is called for in Standard Control between the structure(s) and/or cross-section(s) indicated in the command. ENDCMP indicates the end of a COMPUT series and sends control back to the beginning of Standard Control to start a new runoff problem, based on another storm described in the Executive Control.

Chapter V

PREPARATION OF WATERSHED DATA

The preparation of watershed and storm data for use with the Soil Conservation Service Project Formulation Program (PFP) will now be demonstrated. The example used will be the Duck Creek Watershed near Brockway, Montana and the storm of June 16, 1965. Figure 8 is a map of the Duck Creek drainage showing the location of rain gages and water stage recorders. The stream network is also clearly seen, as is the subdivision of the watershed into subwatersheds.

The largest of the four watersheds under study by the Drainage Correlation Research Project, Duck Creek (54 square miles), located south of Brockway, Montana, was selected for the investigation reported herein. It was chosen for several reasons: 1) its size; 2) the occurrence in 1965 of a rain-caused runoff event which appeared to be suitable for analysis; and 3) particular interest in this watershed by the Montana State office of the SCS. Instruments installed at Duck Creek by the Drainage Correlation Research Project include three weather stations which measure wind velocity, air and soil temperature, and soil moisture; four non-recording and three continuous-recording rain gages; and two continuous-recording water level recorders. One water level recorder is located on the Last Fork of Duck Creek where it is crossed by State Highway 253. The other is located on Duck Creek 330 feet north of the centerline of the county road between sections 11 and 14, T17N, R46E.

The PFP is a generalized program designed to be used on a wide variety of watersheds, and is capable of processing many types of data.

Analysis of the Duck Creek watershed did not require utilization of all the program capabilities. The data supplied to the computer for Duck Creek relate to 1) subwatershed areas and channel lengths, 2) stage-area and stage-discharge relationships, 3) curve numbers and antecedent moisture conditions, 4) time of concentration, and 5) storm data. These are discussed in detail below.

Subwatershed Areas and Channel Lengths

For purposes of constructing a model of the Duck Creek channel network, the Duck Creek watershed was subdivided into twenty-four subwatersheds.

The projected area, channel meander length, and the flood plain length were measured for each of the subwatersheds. Areas were determined by planimetering aerial photographs (scale: 4" = 1 mi.) supplied by the SCS. Both meander and flood plain lengths were measured on the same photographs with a map and plan measure.

Stage-Area and Stage-Discharge Relationships

For the outflow section of each of the subwatersheds, stage-area and stage-discharge curves must be supplied to the PFP. To get this information it was necessary to obtain channel cross-sections in the field at twenty-six locations.

From the cross-section survey notes, stage-area curves were obtained by assuming that the water surface is always laterally horizontal.

The stage-discharge curve for each section was calculated by the Manning equation:

$$Q = \frac{1.49}{n} AR^{0.667} S^{0.5} \quad (33)$$

where n is the Manning roughness coefficient, R the hydraulic radius, and S the friction slope at that section.

Friction Slope: Friction slope is taken to be equal to the channel slope on Duck Creek for several reasons.

First of all, Henderson (1966) states that because of the steep slopes usually present in the upper catchment regions where the runoff problem exists, bed slope will be the only significant slope term and the discharge will be a function of depth alone. Bed slope can be considered the significant slope factor for slopes greater than 10 feet per mile.

Furthermore, the cross-sections on Duck Creek are taken at an average of two miles apart. Also, there are many variations in cross-section shape between any two measured cross-sections. Therefore, the actual friction slope could be expected to deviate from the average bed slope throughout the channel reach. The difference between the average friction slope and the average bed slope, however, would be expected to be very small.

Zone of Transition: The stage-discharge curves used are a combination of curves based on different assumptions. The channel slope used for lower flows, where the flow is confined to the main channel, is the slope of the channel bed. For very large flows covering the flood plain and flowing at depths greater than the vegetation height, the effective slope

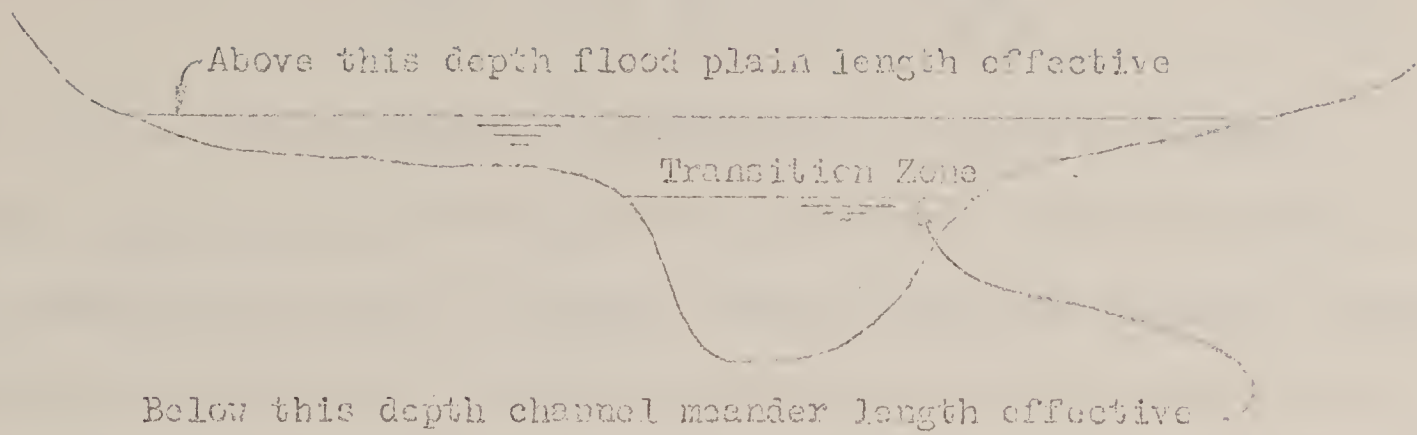
is that of the flood plain. The importance of this consideration of slope is apparent when the ratio of channel length to flood plain length is high, as it is on lower portions of Duck Creek where it reaches a value of two.

A transition zone exists in which the effective slope varies between the channel bed and flood plain slopes (shown in Figure 9). This transition would be difficult to simulate without extensive knowledge of the watershed. Therefore, in this study the stage-discharge curve is a function of the channel bed slope until approximately one foot of water is flowing on the flood plain; above this depth the flood plain slope is considered effective.

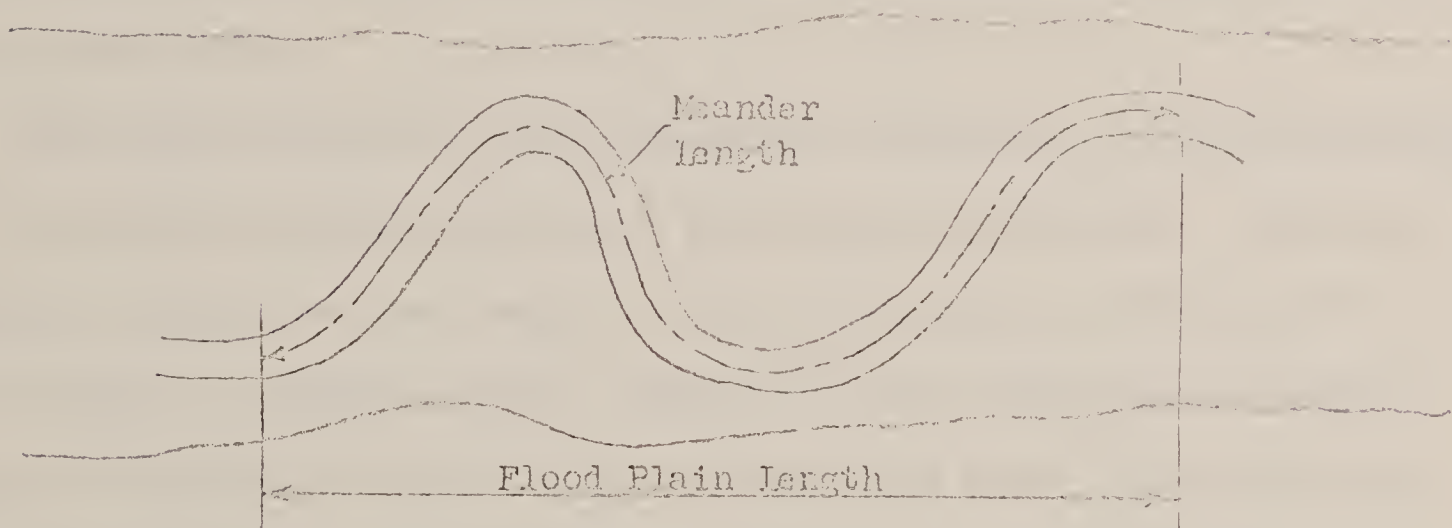
Manning's Roughness Coefficient: The values of Manning's roughness coefficient "n" were selected using a number of photographs of channel sections for which "n" had been calculated by V. T. Chow (1950). These were compared with photographs of Duck Creek to determine the "n" value used for the various channel reaches of Duck Creek.

An attempt was made to check this determination with a set of U. S. Geological Survey (USGS) stereoscopic color slides that have been used for roughness coefficient determinations for that organization. Unfortunately, the set of slides viewed did not include examples similar to those found on Duck Creek.

One further comparison was made in an attempt to evaluate the effective roughness of Duck Creek. This method was developed by Cowan (1956) and can be summarized with the following relationship:



Typical Stream Channel Cross-section



Typical Plan View of Meandering Stream

Figure 9: Typical Plan View and Cross-section of a Meandering Stream

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)n_5 \quad (34)$$

where n is the desired roughness coefficient to be used in Manning's formula. n_0 , n_1 , n_2 , n_3 , and n_4 are the fractional values of n determined respectively by type of channel bed and side slope material, effect of surface irregularities, variation in shape and size of cross-section, effect of obstruction, and vegetation conditions. n_5 is a multiplying correction factor and is a function of the ratio of meander length to flood plain length for any given reach of channel. The roughness coefficient given by this method is larger than those determined by photograph comparisons.

The problem connected with photograph viewing for " n " determinations is that effect of meandering cannot generally be determined. If one is using the channel meander length on slope determinations then the " n " value found is probably correct. However, if the flood plain length is being used (which is the case for overbank flood flows) " n " must be adjusted for the degree of meandering.

For the in-channel flow condition, " n " was determined as discussed above. The meander channel slope was used for this condition. However, for the overbank flow condition, " n " on the flood plain was determined by previously explained methods and was higher than the channel " n " used for the in-channel condition. The slope for the overbank flow is, of course, a steeper one. " n " was taken to be 0.005 higher for the main channel portion of the cross-section than for the overbank flow case. This is

1. The first part of the document discusses the importance of maintaining accurate records of all transactions and activities. It emphasizes the need for transparency and accountability in financial reporting.

2. The second part of the document outlines the various methods and techniques used to collect and analyze data. It includes a detailed description of the experimental procedures and the statistical analysis performed.

3. The third part of the document presents the results of the study. It includes a series of tables and graphs that illustrate the findings of the research. The data shows a clear trend in the relationship between the variables studied.

4. The fourth part of the document discusses the implications of the findings. It highlights the potential applications of the research in the field of finance and economics. It also identifies some of the limitations of the study and suggests areas for future research.

5. The fifth part of the document provides a conclusion and a summary of the key points. It reiterates the importance of the research and the need for further investigation in this area.

6. The sixth part of the document includes a list of references and a bibliography. It cites the works of other researchers in the field, providing a context for the current study.

7. The seventh part of the document contains a list of appendices and supplementary materials. These include additional data, charts, and tables that are not included in the main text.

8. The eighth part of the document is a list of footnotes and endnotes. These provide further details and clarifications on specific points mentioned in the text.

9. The ninth part of the document is a list of acknowledgments. It thanks the individuals and organizations that provided support and assistance during the course of the research.

10. The tenth part of the document is a list of contact information for the authors. It includes their names, addresses, and phone numbers, making it easy for others to reach out to them.

done to account for the fact that much of the flow in the area of the channel is actually following the channel. This is more convenient than assuming that the channel portion of the flow is acting on a milder slope.

Hydraulic Radius: Often the cross-section of a natural stream is so irregular that the Manning equation cannot be conveniently applied to the cross-section as a whole. For the water surface elevations where the flow is in transition between channel and flood plain flow, neither the wetted perimeter P nor the hydraulic radius R accurately represents the actual condition. An alternate way to express this idea is to say that the exponent on R does not equal 0.667 in the transition zone. This is especially true if the main channel is deep and the flood plain is quite flat. To alleviate this problem, the cross-sectional area can be broken up into several portions making the portion divisions at the sharp breaks in the section.

Computer Solution of Manning's Equation: A computer program was written by the author in the Fortran II language for the IBM 1620 computer (Appendix D) to solve Manning's Equation for Q at a given cross-section for all depths (by one-foot increments) between specified limits.

Where the cross-section is irregular, the program first computes area and wetted perimeter for each portion of the cross-section. The stage-discharge information is then printed out for that portion. The same calculations are made and printed out for all the portions of a particular cross-section. Finally, the stage-discharge values for all portions are summed up and printed out as the stage-discharge relationship

valid for the whole cross-section. This summary information is then used as input to the FFP.

Testing the Reliability of Manning's Equation: The reliability of Manning's equation and of the computer program was tested by comparing the stage-discharge rating curves with those which have been developed by the USGS for the East Fork (Figure 10) of Duck Creek and for Duck Creek (Figure 11). The rating curve for the East Fork was developed for the same location (previously described) used in this study. The rating curve for Duck Creek as developed by the USGS is based on their staff gage located at the bridge where Duck Creek flows under the county road between sections 11 and 14, T17N, R46E. The Montana State University stage recorder was originally placed at this same location but was moved in June, 1964 to a point 330 feet north (downstream) of the county road centerline. The old rating curve does not now apply to the new location of the MSU stage recorder and since this paper includes a study of the runoff event of June 16, 1965 a new rating curve had to be developed.

Flow has been measured on three occasions by the USGS since the recorder on Duck Creek was moved. Two of these measurements were made in the spring when the flow was over an iced channel. Therefore, an accurate rating curve cannot be drawn based on actual measurements. It was determined that the Manning equation gave a sufficiently accurate rating curve. This was done by comparing the actual and calculated rating curves at the recorder on the East Fork. At this location, the rating curve given by Manning's equation fell nearly on top of the best visually fitted

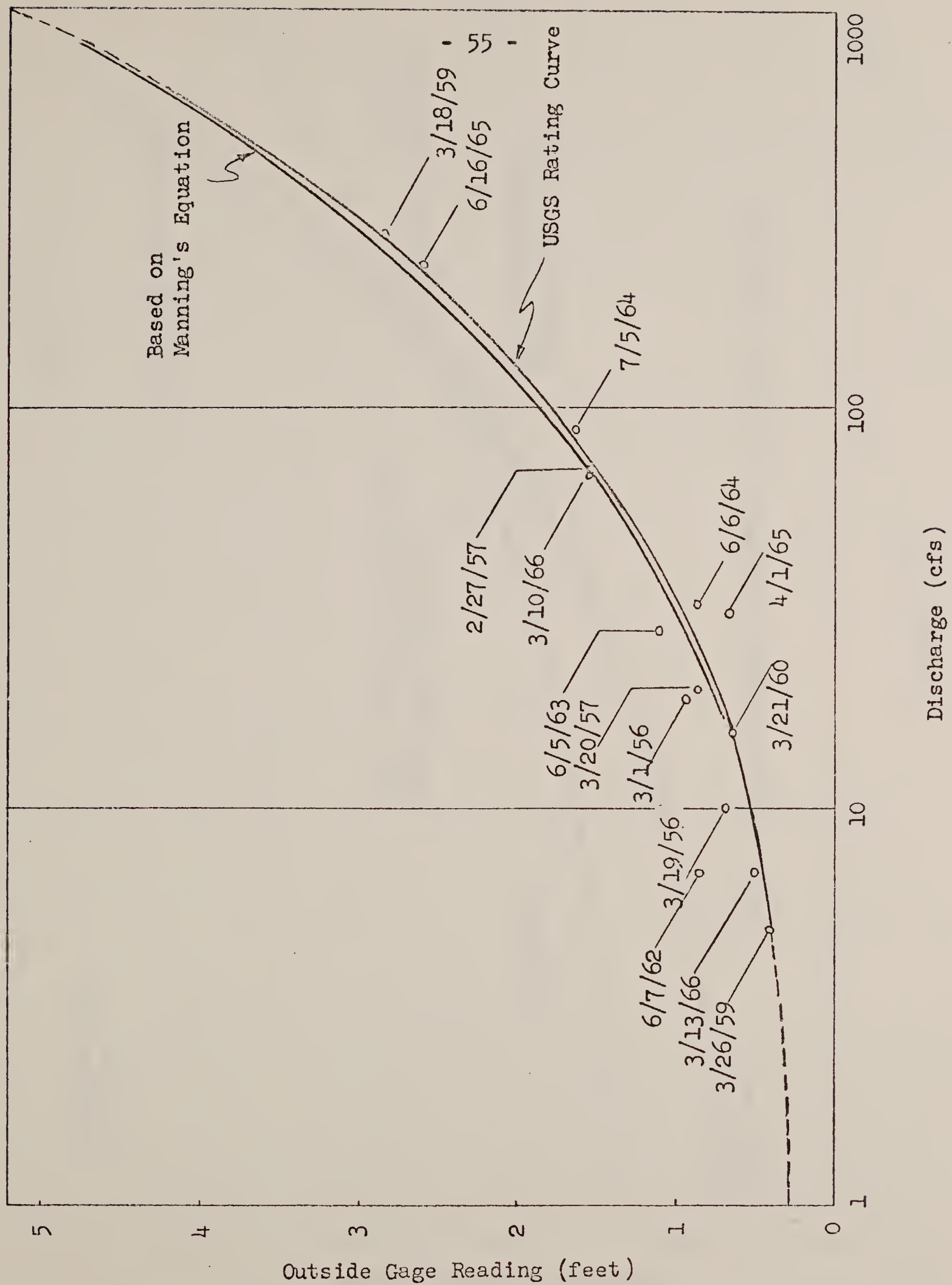


Figure 10: East Fork of Duck Creek Rating Curve



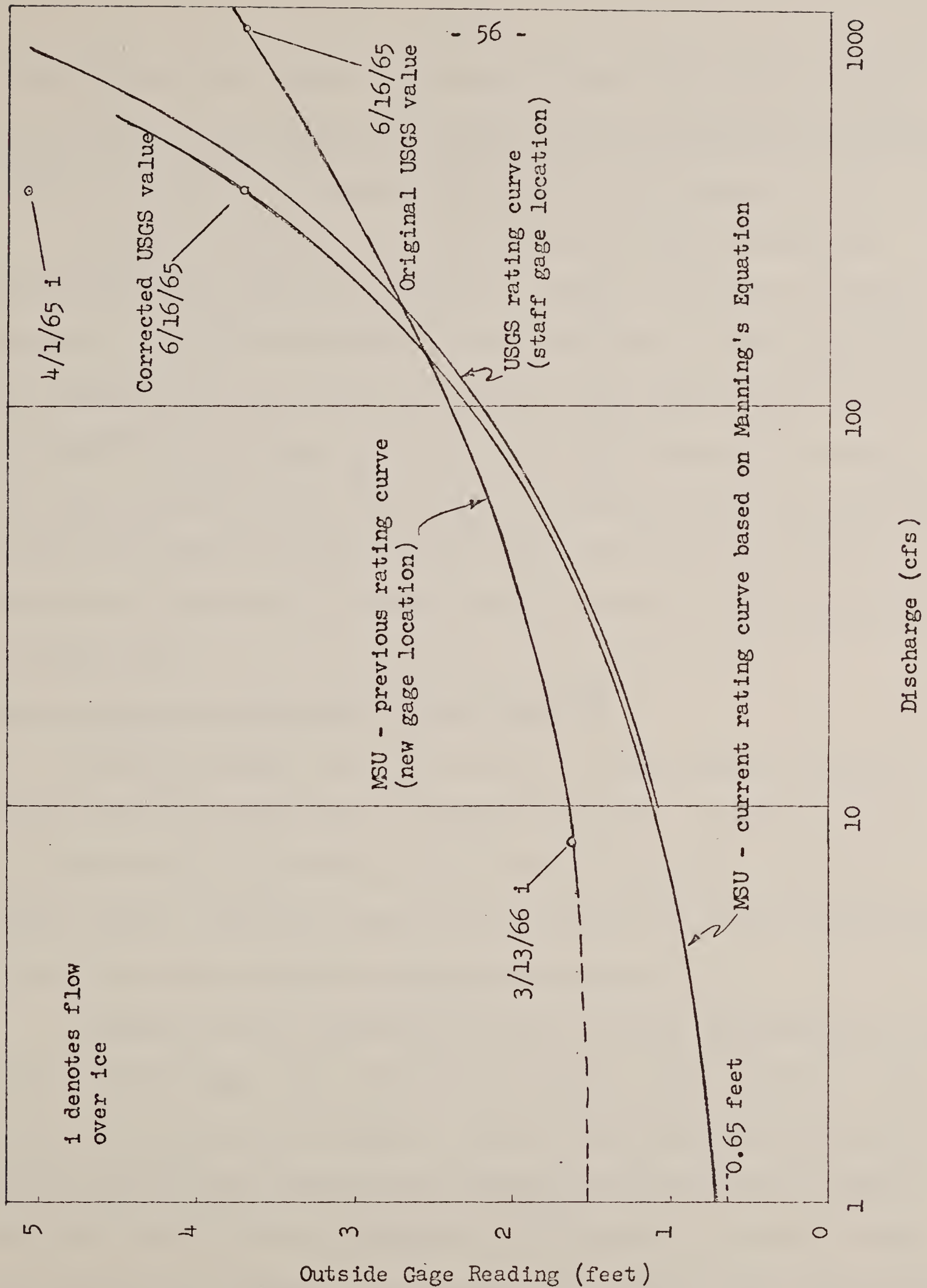


Figure 11: Duck Creek Rating Curve



line through points measured by the USGS. Since this is true, it may be assumed that the rating curve for Duck Creek calculated by the Manning relationship should be adequate for purposes of this study.

As already stated, two different slopes have been utilized for developing rating curves using Manning's equation, depending on whether the flow is mainly in-channel or on the flood plain. In the former case, the length of the main channel centerline or the meander length was used to determine the bed slope. In the latter case, however, the length of the flood plain was used. This length is of course shorter than the meander length giving the floodplain a greater slope than the main channel. In the case of Duck Creek, the average ratio of meander to floodplain length is about 1.89.

Curve Numbers and Antecedent Moisture Condition

As shown in Chapter III, the soil-cover complex number (CN) plays a vital role in the SCS method of determining runoff. A composite CN must be calculated for a watershed if more than one soil type or cover condition exists. The curve numbers used by the SCS for various combinations of soil and cover conditions are given in Appendix C.

A composite CN was calculated for each of the 24 subwatersheds on the Duck Creek watershed. The method used to compute this composite is shown below and is summarized in Table 3. First it was determined what the CN would be if the whole subwatershed consisted of one soil type. This was done on a weighted basis according to the percentages of acres falling in the various cover classes. This was done for each soil type (types B and



(1)	(2)	(3)	(4)	(5)	(6)	(7)
Soil Type	Cover Condition	Curve Number from Appendix C	% of total area	Column (3) x (4)	% of area of soil type	(5) x (6)
B	Strip farming Range	80.5* 69 CN if all type B soil	21.4 78.6	17.2 54.2 71.4	29.3	20.9
C	Strip farming Range	87* 79 CN if all type C soil	21.4 78.6	18.6 62.0 80.6	70.7	<u>57.0</u>
	Composite CN (rounded to nearest whole number) =					78

Table 3: Calculation of Composite Curve Number on a Sample Subwatershed

*An average of CN's for fallow and small grain (straight row in good hydrologic condition) cover condition classes.



C were the only types occurring on Duck Creek).

The weighted CN's obtained for each soil type individually are then again weighted with regard to the percentages of area within each subwatershed that fall into the various soil groups. This method of figuring a composite CN is based on the assumption that the various soil type percentages used occur in those same percentages on each of various cover condition groups used. If this is not true, significant errors in CN calculation could occur. It is felt that such an error would not be important in the case of Duck Creek since in nearly all subwatersheds, one soil type or cover condition was predominant. When this is not true, it is necessary to calculate the weighted CN for each subarea of each subwatershed, whereupon the breakdown of soil types for each subarea would have to be determined. This detail was not considered necessary on Duck Creek.

The determination of CN from Appendix C, which is based on an average antecedent moisture condition (AMC), is input to the PFP. Also, the actual AMC is determined, as described in Chapter III, and read into the PFP. The adjustment of CN is internal to the PFP. For the storm of June 16, 1965 on Duck Creek, the AMC was determined to be dry (Type I).

Time of Concentration

The time of concentration for any given channel reach is taken to be the time of travel for full bank flow conditions. The velocity for full bank flow is determined by Manning's equation at both ends of the channel reach. Travel time is then given by the following relationship:

$$t = \frac{2L}{V_1 + V_2} \quad (35)$$

L is the reach length in feet, V_1 and V_2 are the velocities at the inflow and outflow sections of the channel reach and t is the travel time through the reach. In the case where the reach in question is between a stream cross section and the boundary of the watershed, velocity is taken to be zero at the boundary. These times are then added down through the watershed to find the watershed lag time (time between centers of mass of the rainstorm and the outflow hydrograph) for the watershed. When this calculation does not match the actual lag time as observed from outflow hydrographs, each reach travel time must be adjusted by proportional multiplication so that the actual lag time is obtained. The time of concentration for each individual subwatershed is then taken to be a portion of the stream full-bank flow time plus overland and tributary channel flow times. The combination of main stream and side channel travel times that is longest, is taken to be the time of concentration for the subwatershed.

The tributary channel travel times were based on the velocity of two or three feet per second at the mouth and zero at the head. These two velocities and the length of the tributary channel are then sufficient to give the time of travel as given by equation (35). This is an approximate method but sufficiently accurate on the Duck Creek subwatersheds because they are characteristically long and narrow. The tributary channel

times of travel were short compared to the main channel travel times. For shorter, wider subwatersheds the determination of time of travel and time of concentration would need more scrutiny, for the side channel times would constitute larger percentages of the total times.

Storm Data

Both actual and design storms can be handled by the LFP. The rainfall amount is always read in as either the actual amount from some storm or the design rainfall amount as dictated by some design criteria. The pattern of an actual storm can be taken into account. For instance, when the watershed is broken down into several subwatersheds a different rainfall amount can be designated for each watershed. Temporal variations can also be accounted for in actual storms. The starting time of the rainfall can be different for each subwatershed.

Rainfall patterns can be read into the program as cumulative dimensionless graphs or graphs with the actual time and rainfall amounts as abscissa and ordinate, respectively. As many as nine different patterns can be used in any one run of the computer program.

The amount of rainfall falling on each of the 24 subwatersheds on Duck Creek was determined by the Thiessen Method. The Thiessen polygon was drawn for the seven rain gages on or near the watershed. With this method the rainfall over the entire Thiessen subarea is considered to be uniform and equal in value to the amount recorded at the gage which generally falls in the center of the polygon if the gages are equally spaced. As would be expected, some of the subwatersheds fall in two or three of



the Thiessen polygons, in which case the rainfall amount is a weighted amount based on the percentages of area falling in each of the polygons.

Fortunately, the edges of the polygons fall close to the ridge between the East and West Forks of Duck Creek; this eliminates the largest criticism of the Thiessen method which says that the method does not take into account geographic features.

Storm movement was taken into account by varying the starting time for each of the subwatersheds. The speed of the storm was determined by examination of the records of the three continuous recording rain gages. The time for movement of the storm across the watershed was found to be about 3.5 hours for the storm of June 16, 1965. The starting times for the various subwatersheds were then determined to the nearest 0.1 hour by visually determining the relative positions of their centroids.

Summary

The amount of and kinds of data supplied to the PFP vary from watershed to watershed. Not all types of data that could be processed by the program need be used in every watershed model developed for the computer solution. For example, although there are a number of small stock pond reservoirs on Duck Creek watershed, their effect is felt to be minor, and therefore the model does not include any reservoir information. The data supplied to PFP has been discussed in this chapter. With this information, the PFP is capable of calculating the runoff hydrograph from Duck Creek based on the usual SCS hydrologic formulas and procedures.

Chapter VI

ACTUAL AND HYPOTHETICAL STORMS

The actual storm of June 16, 1965 on Duck Creek was used to adjust a rainfall-runoff model of the watershed. Then hypothetical storms based on U. S. Weather Bureau frequency data were routed through the model to predict peak discharges.

Actual Storm of June 16, 1965

The actual storm used for model adjustment on Duck Creek has heretofore been called the storm of June 16, 1965. The resulting peak discharge actually occurred on this date, although the storm in question started at about 6 a.m. on June 13 and lasted until about 4 a.m., June 16 for a total duration of 70 hours. These times are average times as the storm had slightly varying durations as it moved across the watershed.

The majority of rainfall occurred during three rain bursts approximately a day apart. The duration of the bursts were approximately 3, 1, and 6 hours.

Figure 12 shows the areal distribution of the total amount of rainfall in the form of an isohyetal map. This is based on the records from a total of seven rain gages on or near the watershed.

The storm moved generally in a northwesterly direction down the watershed's main channel. An indication of the storm movement was obtained by studying records of time versus cumulative rainfall amount by percentage for two of the three continuous recording rain gages. These indicated that the difference in time at the two recorder points at which the same percentages of total storm amount had occurred was about two



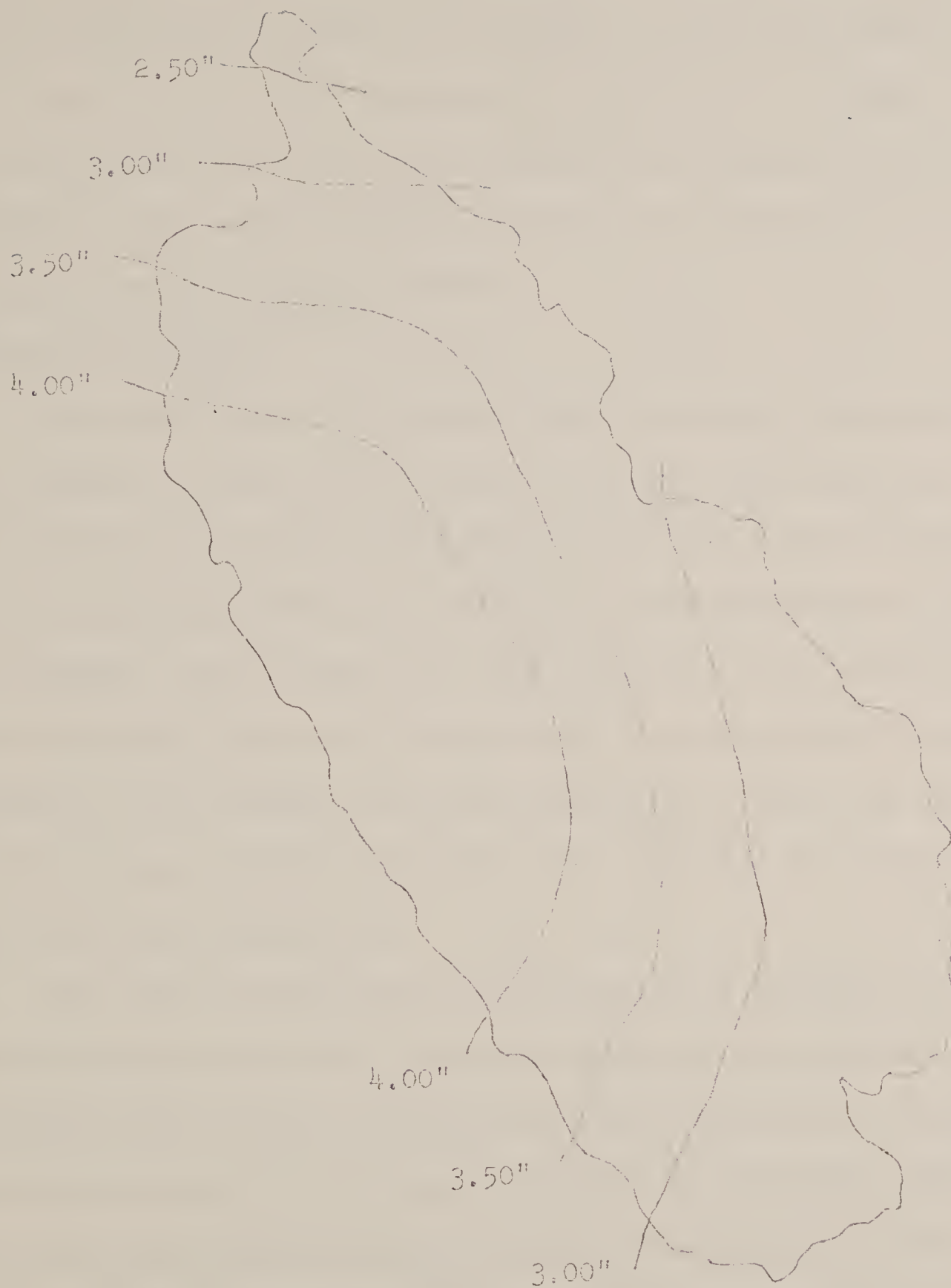


Figure 12: Bathymetric map of June 16, 1965 shown on Lake County, Wisconsin



hours. This was approximately true for all three bursts so that one storm pattern was assured to be adequate for all of the subwatersheds. Assuming that the storm traveled at a constant rate of speed, the 2.0 hour difference in time of storm beginning between the continuous recording points becomes 3.5 hours from the areal centroids of the two most widely separated subwatersheds.

Hypothetical Storms

Storm durations of 12 and 24 hours were chosen arbitrarily for the purpose of using the SCS method to predict peak discharges. Rainfall amounts routed for both durations were in 0.5 inch increments and over a range such that 5, 10, 25, 50, and 100-year frequency storms would be included. These amounts are those predicted by the Weather Bureau in their Technical Paper No. 40 as prepared by David Hershfield (1961). Therefore, for 12-hour storms, precipitation P of 2.0, 2.5, 3.0, 3.5, and 4.0 inches; and 24-hour storms with a P of 2.0, 2.5, 3.0, 3.5, 4.0, and 4.5 inches were routed.

The time of storm beginning was taken to be constant on all subwatersheds since there is little information on the direction of predominant storm movement on Duck Creek. If a consistent pattern of storm movement was known in the case of a particular watershed, then some varied time of storm beginning could be assumed, even for hypothetical storms.

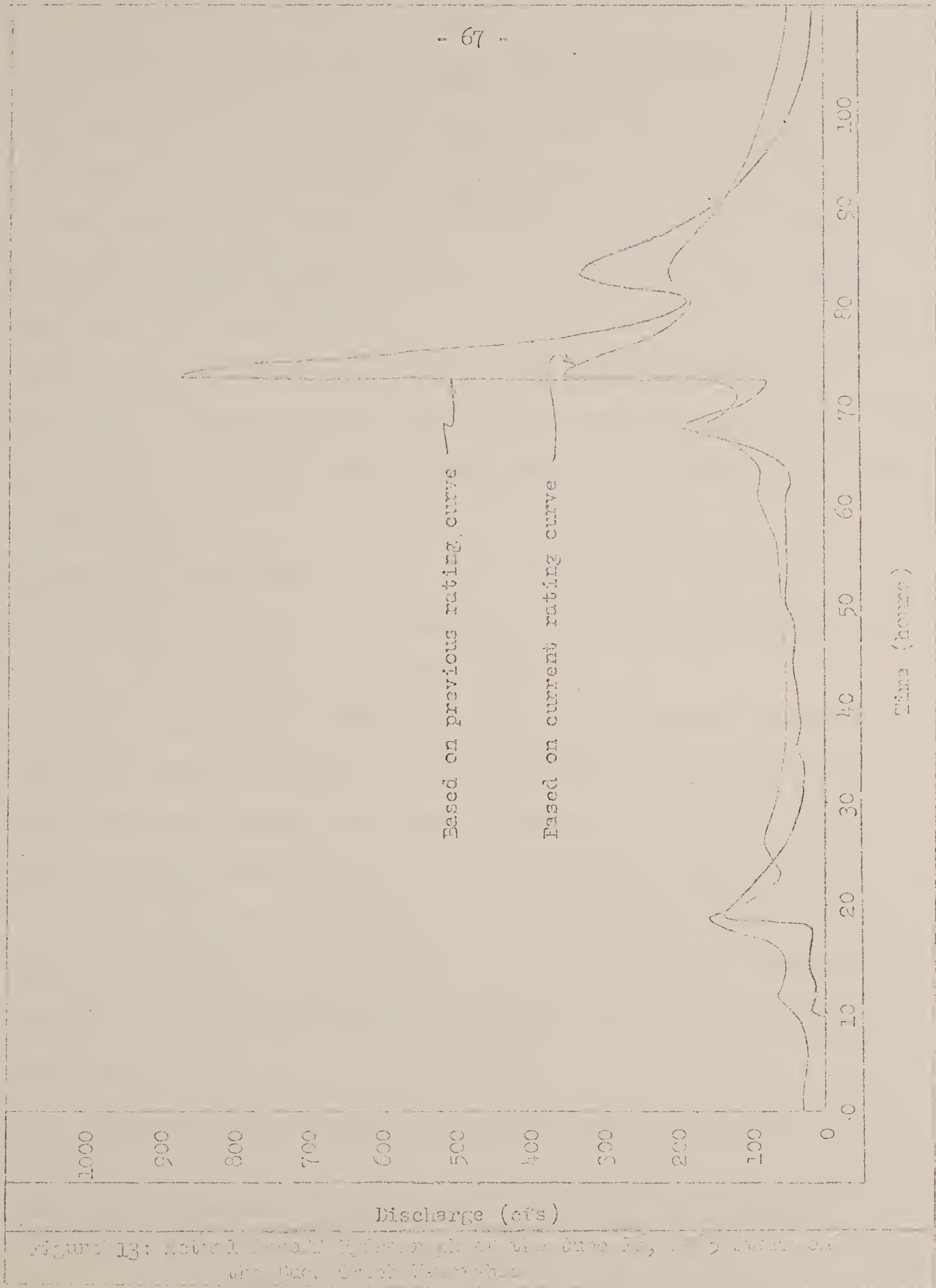
Also, the total amount of rainfall must be assumed to be constant on all subwatersheds unless there is information indicating that the storm center for most known actual storms is consistently located in one

portion of the watershed. Such information is not known for Duck Creek. Therefore, rainfall amounts were considered to be spatially invariant. The effect of the location of storm centers could be studied, however, by centering storms in various parts of the watershed and comparing the calculated outflow hydrograph results.

Actual Runoff Hydrograph

The actual runoff hydrograph for the storm of June 16, 1965 is shown in Figure 13. This hydrograph had been developed by previous investigators based on the best determination of the rating curve in effect at the new stream stage recorder location. The volume of runoff associated with this curve is 0.43 inches for the 54 square mile Duck Creek watershed.

After this study was well under way it was discovered that two errors had been committed in the reduction of the continuous stage recorder data. The first, made by students working on the Drainage Correlation Research Project, was that the volume previously calculated was in error. The resulting yield should have been 0.28 instead of the 0.43 inches. The second error was made by the U. S. Geological Survey in their publication. The peak stage printed for the June 16, 1965 storm was 4.74 feet which gives a peak of 870 cfs based on their rating curve for Duck Creek. The stage value should have been 2.83 feet which gives a peak of 340 cfs instead. Since the original rating curve for the new location was based primarily on this one storm, it is now invalid.



The author has developed* another rating curve based on the Manning Equation, this being justified for reasons described in Chapter V. Only two points are known on the rating curve. One is the storm being routed and the other is the elevation or stage at which the flow is zero. A rating curve was drawn through these two points based on the Manning Equation. The runoff hydrograph was again drawn based on the new rating curve, and is also shown in Figure 13. The ordinates of this plot were read into a computer program written by Gary Lewis to determine the yield. The yield now believed correct for the June 16, 1965 storm on Duck Creek is 0.35 inches.

Computer Runs Completed

A number of runs or storm routings were made on two different occasions. These runs are summarized in Table 4.

Runs 1-A through 1-M and 2-A through 2-M are identical except for the curve numbers (CN) used. The number 1 runs make use of a set of CN's originally thought to be those in effect on Duck Creek. For the number 2 runs, each CN of the set was reduced by the value of three.

The A and B runs (both numbers 1 and 2) caused the June 16, 1965 storm to be routed through the Duck Creek model. The only difference between A and B is the assumption made about the time of the storm's beginning.

Runs C through M are for hypothetical storms. Runs C to H are

*With assistance from Gary Lewis, graduate student at Montana State University working towards an M. S. Degree in Civil Engineering.

Run	Date of run	Type of Storm	Amount (in.)	Duration (hours)	Time of Storm's Beginning	Antecedent Moisture Condition	Curve Number CN
1-A	10/	Actual	Varied	70	Varied	Dry	Originally determined from Appendix C for each subwatershed
B	25/	Actual	Varied	70	Constant	Dry	
C	67	Hypo-	4.5	24	Constant	Average	
D		theti-	4.0				
E		cal	3.5				
F			3.0				
G			2.5				
H			2.0				
I			4.0	12			
J			3.5				
K			3.0				
L			2.5				
M			2.0				
2-A	11/	Actual	Varied	70	Varied	Dry	CN now three less than those for runs 1-A to 1-M
B	27/	Actual	Varied	70	Constant	Dry	
C	67	Hypo-	4.5	24	Constant	Average	
D		theti-	4.0				
E		cal	3.5				
F			3.0				
G			2.5				
H			2.0				
I			4.0	12			
J			3.5				
K			3.0				
L			2.5				
M			2.0				

Table 4: Summary of Computer Runs Completed.



24-hour duration storms with variable rainfall amounts, while I to M are 12-hour duration storms with variable rainfall amounts.

Chapter VII

RESULTS

The data available on the storm of June 16, 1965 on the Duck Creek watershed was used to set up the computer model for analysis by the Soil Conservation Service - Project Formulation Program (SCS-PFP). Two sets of computer runs were made by the SCS in Hyattsville, Maryland on an IBM 360/40 electronic computer.

Actual Storm Evaluation

The initial run gave a total watershed yield of 0.57 inches. This was higher than the actual yield measured at the stream gage which was 0.35 inches. However, at the time the computer runs were made the actual yield was thought to be 0.43 inches (due to previously explained error).

The model was initially set up by evaluating the soil-cover complex numbers (CN) thought to be those in effect on Duck Creek. These are based on the chart used by the SCS (see Appendix C of this paper). The procedure used by the SCS is to check the yield for the originally determined set of CN's. The CN's are then adjusted until the calculated yield equals the actual yield. This is considered to be the calculated runoff hydrograph which can now be compared to an actual runoff hydrograph if it is available.

In an effort to correct the yield to 0.43 inches (thought at the time to have been the actual yield) the author reduced the CN's for all subwatersheds by a value of three and the second set of computer runs was made. The resulting output was hydrographs having yields exactly equal to 0.43 inches.

Computer Results: Figure 14 shows the actual runoff hydrograph for the June 16, 1965 storm, points indicating the computed hydrograph peaks, and two hydrographs which have been postulated on the basis of the best evidence available. The actual hydrograph as measured has a lower, longer profile than the calculated hydrographs. The calculated hydrographs have high peaks and steeper recession limbs.

The four isolated points shown on Figure 14 are the highest peak discharges calculated by the computer runs 1-A, 1-B, 2-A, and 2-B. The first runs, each with a yield of 0.57 inches, are indicated by the two highest points (1-A and 1-B). The higher of these (1-A) is for the case where the time of storm beginning was assumed to vary from one sub-watershed to the next. This hydrograph has a peak discharge of 1416 cfs. The lower of the two (1-B) with a discharge of 1245 cfs is based on the assumption that the starting time was the same for all subwatersheds. The later computer runs with reduced curve number values reduced the yield to 0.43 inches, and the peaks (2-A and 2-B) to 1344 and 1006 cfs respectively. Again the higher peak was for the case of the variable storm beginning.

The two hydrographs shown in their complete form on Figure 14 have been postulated by the author, and are based on the results of the last computer runs. Each of these hydrographs has a yield of 0.35 inches, matching the yield of the actual hydrograph. Their shapes are identical to those of the later computer runs. If further computer runs should be made with lower CN values, it is expected that the shapes of the hydro-

Peak discharge values from results of computer runs:

1500	1-A	Yield = 0.57	Various storm beginning times	1416
1400	1-B	"	"	"
1300	2-A	Yield = 0.43	Various storm beginning times	1245
1200	2-B	"	"	1144

Postulated Runoff Hydrographs
(based on scaled down version of 2-A and 2-B)

Yield = 0.55 Various storm beginning times
Equal

Actual runoff hydrograph
(based on new rating curve)

Discharge (cfs)

Time (hours)

Figure 14: Actual and Postulated Runoff Hydrographs - Paul Creek Watershed

graphs might change slightly. Comparison of the complete hydrographs of the four computer runs which were made indicates that the change in shape would not be great, however.

As can be seen, there is a large discrepancy between the calculated and actual hydrographs with regard to peak. The SCS states that the method was not designed to match the hydrograph in shape but only in peak value. However, in this case, the calculated peak is either 2.41 or 2.74 times as great as the actual peak depending on which assumption is used for the time of storm beginning on each subwatershed. The ramifications of these results will be discussed in Chapter VIII.

Hypothetical Storm Evaluation

One purpose of this study was to predict flood peaks for various frequencies. This was accomplished by routing a number of hypothetical storms through the watershed model to determine the peak discharge predicted by the SCS method. The final results of this portion of the work is summarized in Figure 15. Of course no actual storms have occurred by which these discharges can be compared.

Figure 15 is tentative and should not be used for design purposes. It is shown here only as a demonstration of how the SCS-PFP results of routing of hypothetical storms can be used in conjunction with the U. S. Weather Bureau frequency data to predict peak discharges from small watersheds. The two curves marked WB (for Weather Bureau) are plots of expected precipitation P versus the return period in years (abscissa scale at the bottom of the graph) for the storm of magnitude P for both 24-



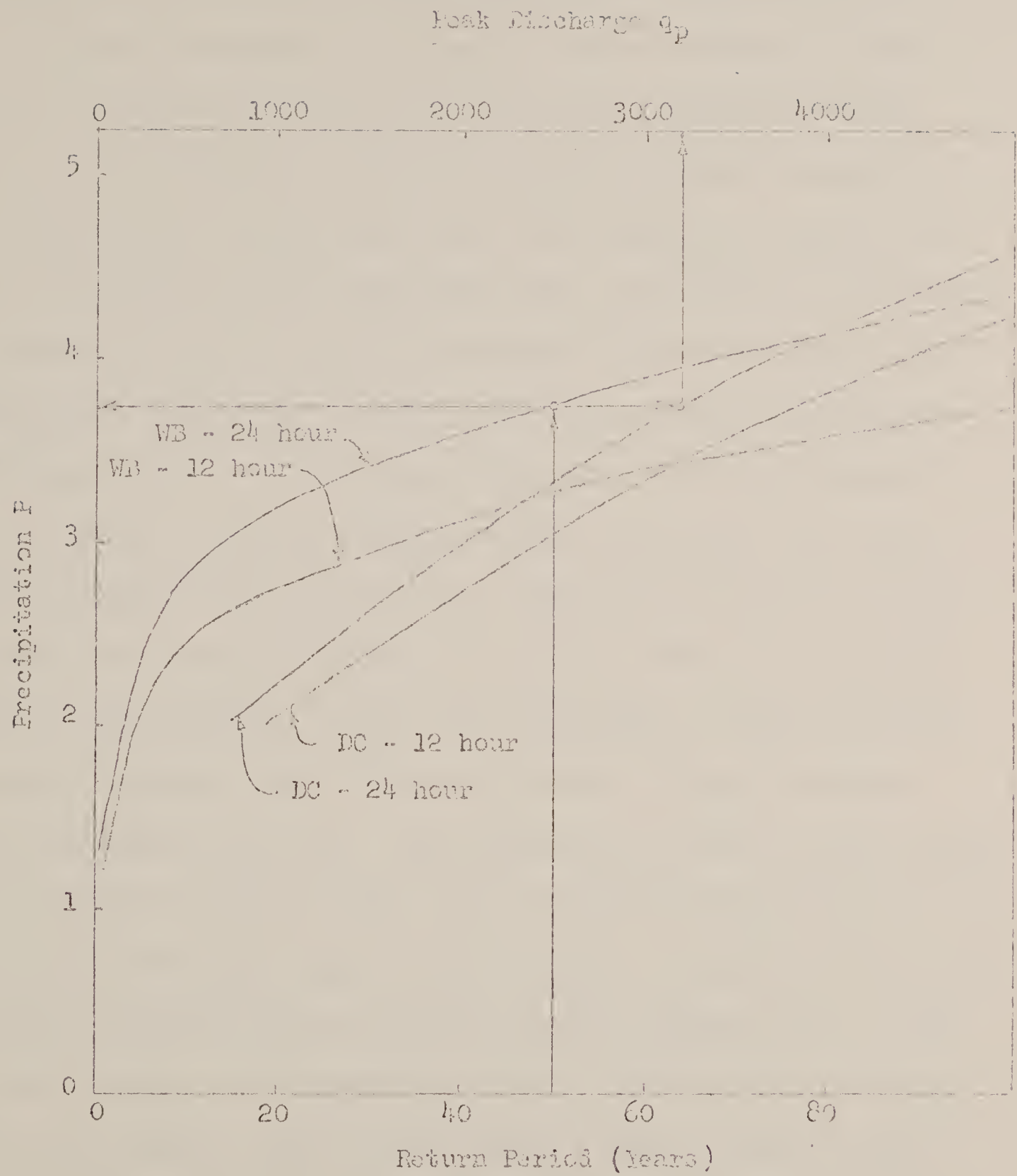


Figure 15: Example of Peak Discharge Design Chart



and 12-hour storms as indicated. The other two curves marked PC (for Duck Creek) show the plots of P versus peak discharge q_p (abscissa scale at the top of the graph) for the Duck Creek watershed as determined by the computer model using SOG hydrologic design criteria for the same two storm durations. These two curves represent the model as set up for the later computer runs (not as yet adjusted as earlier explained and thus not suitable for design use). If additional adjustment of the model could be made, the same procedure as described here would be applicable in setting up a similar set of curves for use in design.

If Figure 15 were to be used for design, the following procedure should be followed to properly predict the peak discharge for a storm. Generally, structures on a watershed are hydraulically designed to function properly for some peak discharge caused by a storm of a particular frequency or return period. Suppose for an example, as shown on Figure 15, we plan to design a culvert that will handle a 50-year frequency storm of a duration of 24 hours. The procedure is to start at the bottom of Figure 15 with the 50-year storm and proceed vertically along the line shown until the WB-24 hour curve is met. Then if the ordinate P is read, we find that the Weather Bureau predicts that the precipitation is 3.75 inches for a 50-year - 24-hour duration storm. If however, we follow a horizontal line (to the right in this example) until we meet the discharge for a 24-hour storm for Duck Creek, and then proceed vertically, we find that $q_p = 3200$ cfs.

On the other hand, if the peak discharge for a runoff event had been



measured on Duck Creek, the exact opposite procedure to that described in the previous paragraph, could be followed to determine the frequency or return period of that storm.

Chapter VIII

Discussion

The investigation described in the preceding chapters has been of two types. First, a model of Duck Creek watershed was developed and then tested by using, as input data to the computer program, information describing an actual storm which occurred on the watershed on June 16, 1965. The resulting synthetic hydrograph generated by the program was compared with the actual hydrograph recorded at the mouth of the watershed. The model was adjusted, after initial comparison, and the program re-run. Second, the Duck Creek model was used to predict the hydrograph from certain design-type storms based on Weather Bureau storm frequency data.

The Storm of June 16, 1965

The results from the computer runs for the June 16, 1965 storm do not compare very closely with the actual hydrograph recorded for the event. The discrepancy is thought to be mainly the result of imperfections in the model, but there is, however, a possibility that the actual hydrograph is not entirely correct. The peak discharge that actually occurred is felt to be accurately known, because it was determined from the USGS rating curve for their crest-stage gage. This rating curve is based on ten current meter measurements over a period of at least ten years. Except for the peak discharge, the rest of the actual hydrograph was determined by relating depths recorded at the water level recorder to discharges. As has been mentioned earlier, the water level recorder is located several hundred feet downstream on Duck

Creek from the crest-stage gage, and the stage-discharge relationships for the two stations are quite different. The best rating curve available at the water level recorder uses Manning's equation and is drawn through only two known points.

Possible imperfections in the model seem to fall in one of the following categories: 1) preparation of the input data and 2) the SCS method.

Preparation of the Input Data: When one is dealing with hydrologic problems the situation never occurs that there is an oversufficiency of data available. There is always a limitation on the time-dependent data available, and an economic limitation on the thoroughness with which the watershed parameters can be obtained. A rather significant amount of watershed data were obtained for the study reported herein, including the results of a medium intensity soil survey, numerous infiltration tests, a complete stadia-level survey of the principal streams, and more than 20 channel cross-section surveys. It would seem that the watershed parameters were delineated, therefore, more completely than could be expected in the typical small watershed design investigation. Results of the study show that even this precision may not be adequate to insure great accuracy in duplication of an actual hydrograph.

The way in which various watershed characteristics were utilized will be discussed below.

The curve numbers (CN) which are used in the SCS program to characterize soil type and land use treatment were developed from large quantities of data which were obtained from all parts of the United States. Since they represent average conditions, and were not developed specifically for Montana, it seems likely that use of the CN table in Appendix C may be responsible for some of the discrepancy in the results. In particular, it seems that the adjustment which is made for dry antecedent moisture conditions (type I) may not be large enough. It may be that for Montana watersheds, the adjustment between average and dry conditions should be larger than that used nationwide.

The CN's used in the first set of computer runs were adjusted for the second set of runs by reducing each number by a value of three. This reduction theoretically should have been proportional to the original value of each CN for each subwatershed. It is uncertain what the effect of reducing in equal instead of proportional amounts might be.

There are indications that the timing of the runoff in the model was in error. This seems to be especially true in the case of the highest peak shown on Figure 14. As can be seen from the main portion of the actual hydrograph, lesser peaks occurred both before and after the maximum discharge. The calculated hydrographs seem to have grouped all three peaks into one and consequently gave too high an estimate for the maximum discharge. The factor that controls the timing on the model is the time of concentration which is determined for each subwatershed. As mentioned earlier, only one time of concentration can be supplied for

each subwatershed, and therefore no variation can be assumed in this time for different discharges. The time of concentration can be shown to vary with rate of discharge so that this becomes a limiting factor in the construction of the model. The times of concentration for this study were based on velocities as calculated by the Manning relationship for full bank flow. The actual event in 1965, produced little or no flooding; it seems probable therefore that the channels were less than bankfull most of the time. The true velocities were probably less than those calculated, and the times of concentration larger than those calculated. This seems likely to account for the fact that the calculated peaks occur at times earlier than the observed peaks.

The SCS Method: The SCS method was developed before electronic computers could be utilized to solve such problems and therefore had to be simply constructed so that it could be solved by hand. And as is true of all methods developed for predicting peak discharges, the SCS method is based on many assumptions. Assumptions are made either because it is thought that they will not greatly affect the solution or because they must be made to make the solution economically possible. Many times in solving hydrologic problems, assumptions must be made because there is a scarcity of information available, even though it is recognized that these assumptions may significantly affect the solution.

The SCS method can be thought of as a mathematical-graphic model which consists of a number of formulae and graphs. Although some flexibility has been built into the operation of the PFP, it appears that the

SCS method has not been basically altered in the conversion into a computer model. The question that then arises is whether the basic method itself could now be made more flexible for computer solutions.

Points in the model that seem to be too rigid are: 1) initial abstraction, 2) lag time, 3) synthetic unit hydrograph shape, 4) storm pattern, and 5) the reservoir and flood routing methods.

The initial abstraction I_a is assumed to be equal to 0.2 S and the lag time to be equal to 0.6 of the time of concentration. The coefficients of 0.2 and 0.6 are certainly for the average conditions for a number of watersheds that were studied by the SCS. The raw data from which these coefficients were determined are not given in the SCS's National Engineering Handbook (NEH). If these data had been supplied, it might be possible to choose more nearly correct factors for a specific area.

Also, the NEH defines only one synthetic hydrograph shape, which is intended to be used in all cases. The PFP, however, is capable of absorbing any unit hydrograph that the user wishes to supply. From this, it would seem that the SCS method as outlined in the NEH is more restrictive than is necessary now that the more flexible computer programs are available.

Only two possible storm patterns are given by the SCS for use in testing hypothetical storms. The pattern most commonly used (shown in Figure 5) is one in which the main burst of the storm comes at about the midpoint of the storm in time. The other has the main storm burst come

earlier in the storm. Again, the PMP, being able to accommodate any storm pattern shape, is more flexible than the method described by the SCS.

The H&H suggests several methods of flood routing which may be used. Only two of these, however, have been incorporated into the computer program. The convex method is the only procedure for channel flood routing available, and the storage-indication method is the only procedure for reservoir flood routing available. These are methods which are easily adaptable to manual solution and perhaps were chosen for this reason. Possibly, other methods would be more realistic and as easily handled by the computer. By simply adding other routing subroutines, the overall computer model could be made more flexible.

Design-Type Storms

An example of design peak discharge determination based on frequency of storm was given in Chapter VII and needs no further amplification here. It might be pointed out, however, that the accuracy with which the peak discharge of a hypothetical storm can be predicted can, be no better than the accuracy of the determination of the amount of rainfall to be expected in any given location for the storm frequency chosen.

The hydrographs which were determined for design storms should be considered only for their qualitative meaning and not their quantitative results. The reason for this, of course, is that the model of the watershed was not thoroughly adjusted in the sense described in Chapter III. Further adjustments in the model or watershed parameters could now be

made quite readily. With a satisfactorily adjusted model available, the hypothetical storms could be routed through the model once more, and design hydrographs would be thereby obtained. As further runoff events occur on Duck Creek, it may become possible to determine how the runoff pattern changes with size of storm; this would permit modifications to be made to the watershed model, which could then again be subjected to the hypothetical storms, and still more definitive results obtained.

Program Use by Montana State Highway Department

The results of the study indicate that before the Montana State Highway Department could reliably use the SCS method of predicting probable peak discharge from Montana watersheds, a long range study and collection of data would first be necessary. Before the SCS method could be applied, maps locating soil types and land use on the watershed, or a single map delineating the soil-cover complex numbers (CC) on the watershed, would have to be made available. Land use maps could be drawn largely with the use of aerial photographs and a minimal firsthand knowledge of the location involved. Hydrologic soil types can presently be determined in many areas in Montana from existing data from SCS soil surveys. However, large sections of Montana have not yet been surveyed. Therefore, a complete soil type map for the state could not now be compiled.

For prediction of a design peak discharge from a particular watershed, additional data would have to be collected. Areas of subwatersheds and channel reach lengths could be measured from aerial photographs if they

were available for the watershed in question. Field surveys would be necessary to determine channel cross-sections. Channel bed slopes would also have to be determined in the field unless suitable topographic maps were available. The Manning roughness coefficient could be determined by inspecting the area or by viewing photographs of the channel.

Montana State University is presently installing a third-generation computer, the SDS Sigma 7, which will be capable of running the current (1967) version of the SCS-IFF with a few modifications. Unfortunately, the Sigma 7 was not available during the time of this study, and it was not feasible to make more than the two sets of runs previously described. To effectively make use of such a program, the hydrologist should have easy access to the computer because of the several trials which seem to be necessary in adjustment and use of a model.

In summary, the SCS method requires collection of a reasonably large amount of data. It has not been possible, with the data which were available for this study, to evaluate the SCS method to determine whether such amounts of data collection are justified.

Chapter IX

CONCLUSIONS

The investigation reported herein has been a study of the SCS method of predicting peak discharge flow rates from rain-caused storms on small watersheds. Data for one actual storm which occurred on Duck Creek in Prairie and McCone Counties, Montana, together with several hypothetical storms for the same watershed, were processed by a computerized solution of the SCS method. The resulting synthetic hydrograph for the actual storm was compared with the actual hydrograph which was recorded for the event.

Findings from the study lead to the following conclusions:

1 - The SCS method is a systematic, logically organized procedure which generates a synthetic hydrograph for a small watershed. It utilizes data characterizing the causative rainstorm, watershed parameters, and accepted flood routing techniques. The method has been effectively programmed by the SCS for computer solution.

2 - Comparison of the synthetic hydrograph generated by the computer program for the storm on June 16, 1965 on Duck Creek with the actual hydrograph which was recorded for the same event, shows considerable discrepancy. This is thought to be primarily caused by using improper values for watershed parameters and flood routing coefficients as input to the computer program. Alternative factors which may explain part of the discrepancy include the possibility that the actual hydrograph for the event may not be entirely accurate, and the possibility that certain tables and graphs utilized by the SCS method may not reflect the proper values which

Chapter X

RECOMMENDATIONS AND SUMMARY

Since the model of Duck Creek watershed has now been constructed and partially adjusted, further adjustment should be made as more runoff events occur on Duck Creek. Also, when additional discharge measurements are made by the USGS, the reliability of the present rating curve at the Montana State University recorder on Duck Creek should be checked.

Models of other Montana watersheds should be constructed to ascertain the variability in runoff characteristics among them. Duck Creek may not be representative of eastern Montana watersheds for some unforeseen reasons. Thus, with other models available, the errors in design storm peak discharge predictions would be diminished.

Another area of potential study would be the testing of the various facets of the SCS method. Since the present program is based largely on the method used before the advent of the digital computer program, there are components of the method that could be more sophisticated without making the problem solution more difficult. It would appear that the synthetic hydrograph shape could be made more flexible to account for differences in area, shape of watershed, climatic condition and location. Calculation of the theoretical time of concentration and lag time needs further study. The importance of synthesizing the timing of runoff has been shown by the example used herein.

In order to make further study both easier and more profitable, the SCS-PFP should be altered slightly for use on the new digital computer presently being installed at Montana State University.

Summary

Prediction of the peak discharge of a rain-caused runoff event from a watershed by the method developed by the SCS has been discussed. The development of this method was outlined in Chapter III.

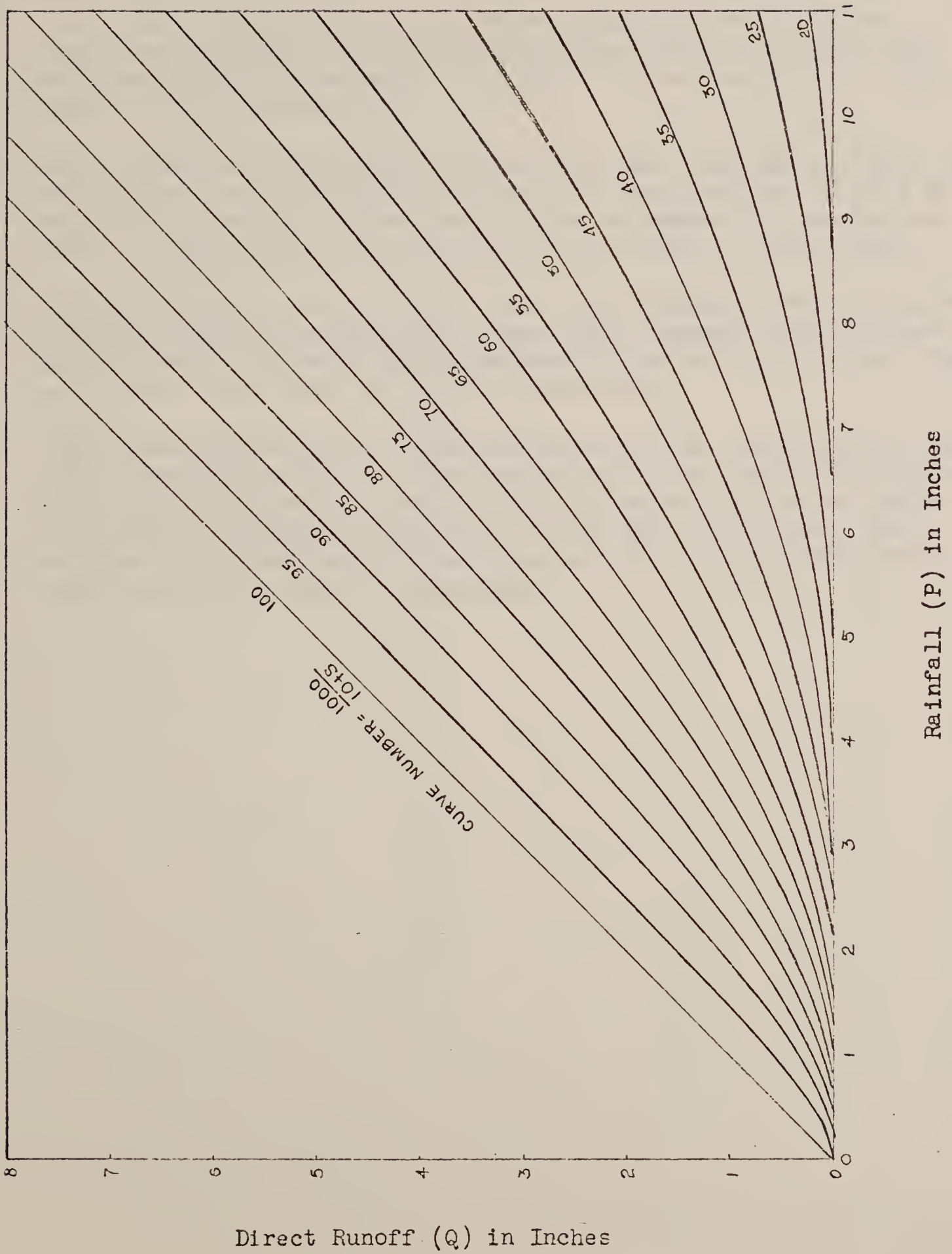
More recently the method has been programmed for use with an electronic computer. A description of this program and an explanation of its operation is found in Chapter IV.

The ability of the SCS method to accurately predict peak discharge was partially tested with use of the computer. The model of the Duck Creek watershed was constructed and was subjected to the storm of June 16, 1965. Although the test run is not sufficient to evaluate the predicting ability of the program, the model is now available for Duck Creek and can be further adjusted by testing other storms.

APPENDIX

APPENDIX A

Solution of Runoff Equation



APPENDIX B

Hydrologic Soil Types

- A. (Low runoff potential). Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission.
- B. Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.
- C. Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.
- D. (High runoff potential). Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

APPENDIX C

Runoff curve numbers for hydrologic soil-cover complexes

(Antecedent moisture condition II, and $I_a = 0.2 S$)

Land Use	Cover		Hydrologic soil group			
	Treatment or practice	Hydrologic condition	A	B	C	D
Fallow	Straight row	----	77	86	91	94
Row crops	"	Poor	72	81	88	91
	"	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	"	Good	65	75	82	86
	Contoured & Terraced	Poor	66	74	80	82
	"	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	Contoured & Terraced	Poor	61	72	79	82
		Good	59	70	78	81
Close-seeded legumes <u>1/</u> or rotation	Straight row	Poor	66	77	85	89
	"	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	"	Good	55	69	78	83
Meadow	Contoured & Terraced	Poor	63	73	80	83
	"	Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	"	Fair	25	59	75	83
	"	Good	6	35	70	79
Meadow		Good	30	58	71	78
Woods		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		----	59	74	82	86
Roads (dirt) <u>2/</u> (hard surface) <u>2/</u>		----	72	82	87	89
		----	74	84	90	92

1/ Close-drilled or broadcast.

2/ Including right-of-way.

APPENDIX D

MANNING Q CALCULATION FROM X-SECTION NOTES

C
C A = AREA BETWEEN TWO X-SECTION POINTS
C AXSEC = TOTAL AREA OF ENTIRE X-SECTION
C ATOTL = TOTAL AREA OF PORTION OF X-SECTION
FOR EXAMPLE, AREA OF LEFT FLOOD, CHANNEL,
OR RIGHT FLOOD PLAIN
C COEFN = MANNING N COEFFICIENT
C DELWS = DISTANCE Y TO WATER SURFACE FROM NEXT POINT
C (X,Y) ABOVE WATER SURFACE
C IHWSEL = HIGHEST WATER SURFACE TO BE INVESTIGATED
C LOWSEL = LOWEST WATER SURFACE TO BE INVESTIGATED
C NOWSEL = NO. OF WATER SURFACE ELEVATIONS TESTED
C NOXSA = NO. OF X-SECTION AREAS
C NOXSP = NO. OF X-SECTION POINTS, 10 IS THE MAXIMUM
C NSUBXS = NEW SUB X-SECTION
C IF = 0 NEW SUB-SECTION TO BE READ
C IF = 1 NEW X-SECTION TO BE READ
C P = PERIMETER
C PTOTL = TOTAL PERIMETER IN SUB X-SECTION
C PXSEC = PERIMETER OF WHOLE X-SECTION
C SAVXL = LOCATION TO SAVE VALUE OF X ON LEFT SIDE OF WS
C INTERSECTION TO BE USED WITH NEW WS ELEVATIONS
C SAVXR = SAME AS SAVXL EXCEPT FOR RIGHT SIDE
C SLOPE = CHANNEL SLOPE
C Q = TOTAL DISCHARGE IN SUB X-SECTION
C QXSEC = DISCHARGE FROM WHOLE X-SECTION
C VXSEC = AVERAGE VELOCITY FOR WHOLE X-SECTION
C WSELEV = WATER SURFACE ELEVATION
C X(I) = DISTANCE FROM HORZ CONTROL
C Y(I) = ELEV. ABOVE DATUM

C DISCHARGE Q CAN BE CALCULATED FROM MANNINGS EQUATION FOR
C IRREGULAR NATURAL STREAM CROSS-SECTIONS. THE WHOLE X-SEC
C MAY BE SUBDIVIDED INTO SUB-SECTIONS AT SHARP GEOMETRICAL
C BREAKS. THERE ARE FOUR DATA CARDS NECESSARY FOR EACH SUB-
C SECTION. THERE FORMAT IS AS FOLLOWS.

C	CARD	COLUMNS	
C	1	1 - 80	PRINT ANY TITLE SHOWING WATERSHED NAME, LOCATION, SECTION NUMBER, ETC.
C	2	1 - 5	NOXSP RIGHT JUSTIFIED
C		6 - 10	LOWSEL RIGHT JUSTIFIED
C		11 - 15	IHWSEL RIGHT JUSTIFIED
C		16 - 20	NSUBXS RIGHT JUSTIFIED
C		21 - 30	COEFN IN FORMAT XXXX.XXXXX
C		31 - 40	SLOPE IN FORMAT XXXX.XXXXX
C	3	1 - 8	X DIST TO 1ST POINT FROM REFERENCE WITH FORMAT XXXXX.XX
C		9 - 16	X DIST TO 2ND PT FROM REFERENCE PT
C		17 - 24	3RD
C			ETC.
C	4	1 - 8	ELEVATION Y CORRESPONDING TO X OF CARD 3
C			ETC.

C AFTER ALL SUB-SECTIONS HAVE RUN FOR A GIVEN X-SECTION
C ANOTHER CARD MUST BE READ TO INITIALIZE THE SUMMARY PRINT
C OPERATION. THE CARD MUST HAVE THE LOWEST VALUE OF LOWSEL
C FOR THE X-SECTION IN COLUMN 1-5, RIGHT JUSTIFIED.


```

        DIMENSION TITLE(40),X(10),Y(10),DXDY(9),A(9),P(9),D(10),
        1VXSEC(600),AXSEC(600),PXSEC(600),QXSEC(600)
        PRINT 104
104  FORMAT(60H0          MANNING Q CALCULATION FROM
        1X-SECTION NOTES
C     NOTE THAT THE FOLLOWING DO LOOP IS GOOD ONLY FOR ELEVA-
C     TIONS FROM 2500 TO 3100 FEET  CHANGE ELEV. IN STATEMENT
C     19 TO FIT WATERSHED,DIMENSION STATEMENT MUST ALSO BE
C     CHANGED FOR VXSEC, AXSEC, PXSEC, AND QXSEC.  CONSTANTS
C     IN STATEMENTS 24, 106, AND 107 MUST CONFORM
19  DO 23 KK=2501,3100
24  K=KK-2500
        AXSEC(K)=0.0
        PXSEC(K)=0.0
23  QXSEC(K)=0.0
        1 READ 89,(TITLE(I),I=1,40)
89  FORMAT(40A2)
        PRINT 90,(TITLE(I),I=1,40)
90  FORMAT(1H1,40A2///)
        READ 91,NOXSP,LOWSEL,IHWSEL,NSUBXS,COEFN,SLOPE
91  FORMAT(4I5,2F10.5)
        IF(NOXSP) 16,16,2
        2 READ 92,(X(I),I=1,NOXSP)
        READ 92,(Y(I),I=1,NOXSP)
92  FORMAT(10F8.2)
        PRINT 93,(X(I), I=1,NOXSP)
93  FORMAT(4H X= 10F8.2/)
        PRINT 103,(Y(I),I=1,NOXSP)
103  FORMAT(4H Y= 10F8.2//)
        NOXSA=NOXSP-1
        PRINT 95, COEFN
95  FORMAT(9H0      N  =F10.5/)
        PRINT 97,SLOPE
97  FORMAT( 9H SLOPE  =F10.5///)
        PRINT 101
101  FORMAT(62H WATER SURFACE  DISCHARGE Q  END AREA
        1 VELOCITY  PERIMETER  /61H  ELEVATION  (CFS)
        2 (SQ FT)    (FPS)      (FT)    //)
        DO 15 J =LOWSEL,IHWSEL
        ATOTL=0.0
        PTOTL=0.0
        DO 3 I=1,NOXSP
        AJ=J
        D(I)=AJ-Y(I)
        IF (D(I)) 4,3,3
        4 D(I)=0
        3 CONTINUE

```



```

DO 18 I=1,NOXSA
  II=I+1
  IF(Y(II)-Y(I)) 5,17,5
17 DXDY(I)=0
  GO TO 18
  5 DXDY(I)=(X(II)-X(I))/(Y(II)-Y(I))
18 CONTINUE
C   CALCULATE X AND Y COORDINATES WHERE W.S. INTERSECTS
C   X-SECTION ON LEFT SIDE FACING DOWNSTREAM.  THE IF(DXDY)
C   BYPASSES THIS SIDE IF X-SECTION IS OF RIGHT FLOOD SEC-
C   TION ONLY
  K=1
  IF(DXDY(K)) 6,81,81
81 SAVXL=X(K)
  KK=K
  GO TO 9
  6 K=K+1
  IF (D(K)) 6,6,7
  7 KK=K-1
  SAVXL=X(KK)
  DELWS=Y(KK)-AJ
  IF (DELWS) 9,9,8
  8 X(KK)=X(KK)-DELWS*DXDY(KK)
C   CALCULATE X AND Y COORDINATES WHERE W.S. INTERSECTS
C   X-SECTION ON RIGHT SIDE FACING DOWNSTREAM
  9 L=NOXSP
  LLL=L-1
  IF(DXDY(LL)) 82,82,10
82 SAVXR=X(L)
  LL=L
  L=LLL
  GO TO 13
10 L=L-1
  IF (D(L)) 10,10,11
11 LL=L+1
  SAVXR=X(LL)
  DELWS=Y(LL)-AJ
  IF (DELWS) 13,13,12
12 X(LL)=X(LL)-DELWS*DXDY(L)
C   CALCULATE AREA, PERIMETER, AND MANNING Q
13 DO 14 I=KK,L
  II=I+1
  A(I)=(ABS(X(II)-X(I)))*((D(II)+D(I))/2.)
  P(I)=SQRT(((ABS(X(II)-X(I)))**2.)+(ABS(D(I)-D(II)))
  1**2.))
  ATOTL=ATOTL+A(I)
14 PTOTL=PTOTL+P(I)

```



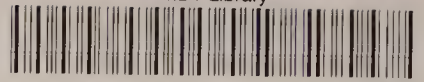
```
      Q=(1.49/COEFN)*((ATOTL**1.667)/(PTOTL**0.667))*(SLOPE
1**0.5)
      AVEVEL=Q/ATOTL
      X(LL)=SAVXR
      X(KK)=SAVXL
      PRINT 100, J,Q,ATOTL,AVEVEL,PTOTL
100  FORMAT(3X,I5,8X,F10.0,3X,F8.1,3X,F6.2,6X,F6.1)
106  L=J-2500
      PXSEC(L)=PXSEC(L)+PTOTL
      AXSEC(L)=AXSEC(L)+ATOTL
      QXSEC(L)=QXSEC(L)+Q
      15  VXSEC(L)=QXSEC(L)/AXSEC(L)
      IF(IHSUBXS)20,1,20
      20  PRINT 105
105  FORMAT(38H1      SUMMARY FOR ENTIRE CROSS SECTION  ///)
      PRINT 101
      READ 22, LOWSEL
      22  FORMAT(I5)
      DO 21 J =LOWSEL,IHWSEL
107  L=J-2500
      21  PRINT 100,J ,QXSEC(L),AXSEC(L),VXSEC(L),PXSEC(L)
      GO TO 19
C     PLACE TWO BLANK DATA CARDS AT END OF DATA
      16  CALL EXIT
      END
```


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